Master Thesis, Department of Geosciences

Interpretation of CPTu results in silty clay and assessment of strength parameters

Geotechnical investigation of silty clay at Gunnestad, Sande municipality, Norway

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Abstract

The clay zone at Gunnestad area, which is located in Sande municipality in south-east of Norway, has high risk with respect to slope stability. The risk of slope instability is associated with silty clay sediments having the property on the border between clay and silt. As a result there were uncertainties in reliable estimation of strength parameters and undrained shear strength for slope stability analyses. The main purpose of this study was to investigate the mechanical properties of silty clay for reliable slope stability analyses that includes the undrained shear strength, effective friction angle (ϕ'), attraction (a), and deformation and consolidation characteristics. This thesis met the aim by interpreting a number of CPTu (cone penetration tests with measurement of pore pressure) with the support of sensitivity data, plasticity data, and water content from the samples, borehole profiles and pore pressure measurements using piezometers. For reliable evaluation of the mechanical properties of silty clay, CAUC triaxial tests and oedometer tests were done for soil samples from Gunnestad.

Soil types were identified with continuous measurement of cone resistance, sleeve friction and pore pressure for all sites. The Robertson 1990 method was found to be the best method to identify the soil types that showed substantial agreement with soil behaviour type index (I_c) and some of the borehole data. The overconsolidation ratio (OCR) of the area that affects the engineering property of soils was determined for five sites (including the area of interest, Gunnestad) using different methods. The results indicated that soils in the area of interest are slightly overconsolidated. OCR estimated from Q_t (normalized cone resistance) is more reliable as compared to OCR estimated from excess pore pressure (Δu) and pore pressure ratio (B_q). OCR based on former elevation gave similar results to Qt-based OCR. For more accuracy on OCR at Gunnestad, Casagrande's method was applied for interpretation of the oedometer test results.

The undrained shear strength (s_u) based on cone factors (N_{kt}, $N_{\Delta u}$) and SHANSEP model (Stress History and Normalized Soil Engineering Properties which comprises the vertical effective stress and overconsolidation ratio) was analyzed for clay and silty clay. s_u based on $N_{\Delta u}$ and SHANSEP seems more reliable than N_{kt}-based s_u for clay soils. Recommended trend on s_u of silty clay at Gunnestad were based mainly on CAUC triaxial test result at 4% strain rate and N_{kt}-based s_u seems more reliable than s_u based on $N_{\Delta u}$. SHANSEP-based s_u with OCR from Casagrande's method gave also good results. N_{kt} was back-calculated using the reference s_u found from laboratory tests and gave higher values at different depths. Reliable determination of friction angle (ϕ') and attraction (a) for silty clay were found from CAUC triaxial test. Similar result with slightly higher value of friction angle was also estimated from CPTu data. Constrained modulus (M) estimated from CPTu data gave a range of values that vary from 4 to 12 MPa. However oedometer test results at two specific depths showed similar results with those estimated from CPTU data. Variation in coefficient of permeability and constrained modulus with depth for silty clay and other types of soils give several orders of magnitude on coefficient of consolidation.

Contents

List of symbols and abbreviations

Symbol	Description	Symbol	Description
q_c	Uncorrected cone resistance	A_s	Friction sleeve surface area
CPTu	Cone penetration test with pore pressure measurement	USCS	Unified soil classification system
f_s	Sleeve friction uncorrected	I_c	SBTn index
OCR	Overconsolidation ratio	σ’p	Maximum past effective consolidation stress
a	Unequal area ratio	σ'_{vo}	Effective overburden stress,
q_t	Corrected cone resistance	σ_{vo}	Total overburden stress
f_t	Corrected sleeve friction	s_u	Undrained shear strength
u_3	Excess pore pressure measured at upper end of the friction of the sleeve	Δu	Excess pore pressure
u_2	Excess pore pressure measured at lower end of the friction sleeve	u_m	Measured penetration pore water pressure
A_{sb}	Cross sectional area for the top sleeve	u_1	Pore water pressure on the cone
A_{st}	Cross sectional area for the bottom sleeve	u_2	Pore water pressure behind the cone
f_T	Corrected sleeve friction	u_0	Hydrostatic pore water pressure
CPT	Cone penetration test	γ_{water}	unit weight of water
R_f	Friction ratio	q_e	Effective cone resistance
SBT	Soil behavior type	N_{ke},	Cone factors based on effective cone resistance
SBTn	Normalized soil behavior type	N_{kt},	Cone factors based on corrected cone resistance
Q_t	Normalized cone resistance	$N_{\Delta u}$	Cone factors based on excess pore pressure
F_r	Normalized friction ratio	ϕ’	Friction angle
B_q	Pore pressure ratio	γ_{sat}	saturated unit weight of soil

1. INTRODUCTION

1.1 Background

For several years cone penetration test (CPT) that measures cone tip stress (q_c), sleeve friction (f_s), has been extensively used for soil profiling and in geotechnical design through continuous measurements of cone resistance and sleeve friction taken when the cone is pushed at a standard penetration rate of 20mm/s. Its simplicity, repeatability, accuracy and continues record are the most significant advantages of CPT (Lunne, et al. 1997; Robertson, 2009).

CPT was then developed as CPTU (cone penetration test with measurement of pore pressure) in the late 1970's and has now become the most common in situ testing tool for determining undrained strength of clay formations, stress history, dissipation tests, coefficient of consolidation, permeability and moduli values (Karlsrud, et al. 2005; Senneset, et al. 1989). Drained strength of sands can also be determined easily from CPTu data. However, it should be mentioned that correlations for estimation of density and moduli of sands are approximate and should be used as a guide where density corrleation can be improved if the compressiblity of sand is examined from grain charachterstics (Robertson and Campanella, 1983)

It is generally accepted that undrained penetration occurs in clay while drained penetration occurs in sand with standard penetration rate of 20mm/s. In practice, CPTu interpretations are based on empirical correlations between soil properties and CPTu measurements. However, more silty soils having permeability between 10^{-6} to 10^{-3} cm/s experience only partial drainage during penetration by piezocones at this rate (Lunne et al. 1997). Even though silty clay has permeability between 10^{-9} to 10^{-7} cm/s, it is mentioned that soils of transitional type such as clayey sands and silts, silty clays, silts and many residual soils can be conducted under conditions of partial consolidation (Schneider, et al. 2008; Tonni and Gottardi 2011). This implies that interpretation of properties of silty clay can be uncertain both in terms of assessment of soil properties and their identification when penetration occurs under conditions of partial drainage. It is also uncertain to what extent the pore pressure affects the cone resistance. Therefore, it is obvious that clays and sands that exhibit fully undrained and drained characteristics respectively during penetration have no little uncertainty in CPTu interpretations for the estimation of soil parameters that will be used for engineering design. However, soils that undergo partial drainage during penetration test should need careful studies.

1.2 Problem Statement

It is clear that interpretation of clean sand or clay may not work for intermediate soils such as silty clay and silty soils with partial drainage under penetration rate of 20mm/s. Grain size distribution and clay content of silty and silty clay soils are important factors to consider in classifying the behaviour of those soils as clean clay, clean sand or intermediate one.

Interpretation of soil strength for engineering design can be correctly estimated if it involves undrained loading being penetration of the cone is also undrained or on the other hand fully drained loading being penetration of the cone is fully drained. However, there is difficulty for determining soil strength interpretation for partially drained type that occur as drained in large time scale which can cause design problem (Lunne et al. 1997; Schneider et al. 2008). It is not only the behaviour of the soil that causes design problem but the penetration rate can also affect the drainage condition and the value of cone resistance which is important element for defining soil parameters (Schneider et al. 2008).

Particular problem based from previous studies made by Norwegian Geotechnical Institute (NGI, 2006 and NGI, 2011), is that some sites in the municipality of Sande , south east of Norway, consist mainly of low to moderately sensitive silty clay to clay silt where strength parameters are difficult to interpret for those types of soils. On the basis of previous risk assessment conducted by Norwegian water resources and energy directorate (NVE), the zone 502, Gunnestad area considered to have hazard level “medium” and consequence class “very serious” resulting in risk “4”. This requires additional investigation. So the main significance of this study is hopefully to contribute to determining reliable estimation of strength parameters for the conservative estimate of stability calculations. In addition to difficulty in interpretation, silty clay has also low compressibility characteristics and most of the structure constructed on it can be affected especially in stabilization and settlement. To prevent this from occuring, the engineering properties of silty clay must be determined before the design work starts.

1.3 Research Objectives

The main target of the research is to determine the mechanical properties of silty clay. The study is based mainly on 10 CPTu soundings from different areas within Sande municipality. For reliable interpretation, previous drilling data for some sites and laboratory tests for silty soils were analysed. The study has to address the questions listed below:

- Determination of undrained shear strength of silty clay based from mechanical properties of clay and silty soils
- What would be the reliable value of strength parameters such as effective friction angle (ϕ') and attraction (a) which are main elements for slope stability calculation
- How the deformation and consolidation characteristics behave for silty clay soils

In order to achieve the target exact soil type has to be identified first, because wrong interpretation of soil type can be problematic in calculation of slope stability design parameters. Determination of overconsolidation ratio (OCR) for silty clay and other soil types is also one of the individual objectives. In addition, CPTu soundings are correlated with sea level.

1.4 The Site

The study area, Sande municipality is found in the south eastern part of Norway and the locations of the 9-CPTu tests are shown in fig-1 with coordinate list mentioned in table-1. Detail map of the study area is found in appendix-1. As cited in NGI (2011) report; this area was previously studied by NGI and private consultants (Grunn-Teknikk (1980), NGI (2001), Grunn-Teknikk (2005), NGI (2006), Multiconsult (2009) and Cato Geotechnikk (2010)) for assessing stability condition in the area.

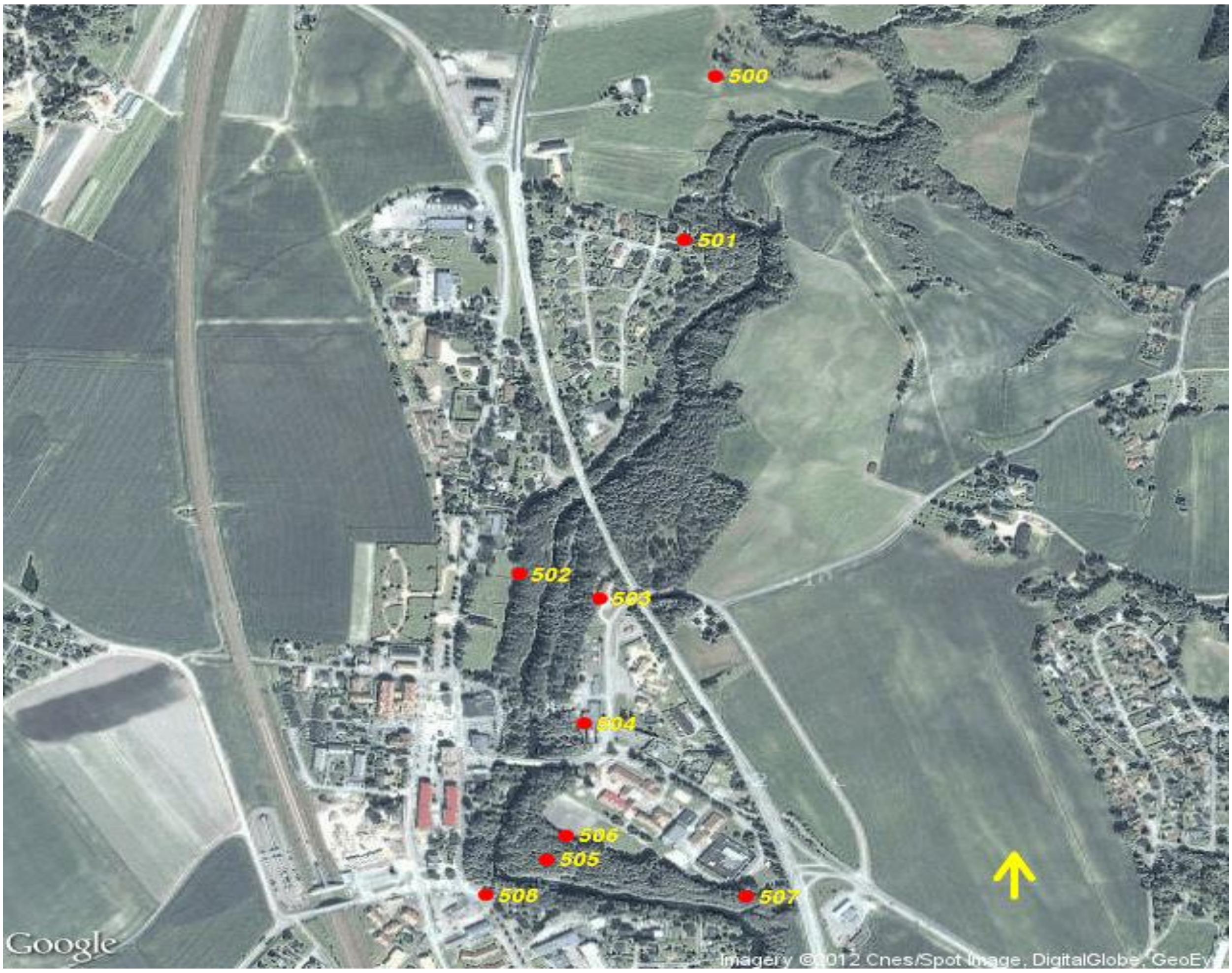


Figure 1 Location of the study area.

According to Norwegian Geotechnical Institute (NGI) reports of 2006 and 2011, the soil deposit in downtown Sande consists largely of low to moderately sensitive silty clay to clay silt deposit and layers of silt and sand are local variations. Clays were also found by the river below the municipal premises. Some ground investigation was carried out from May-October, 2011 by "GeoStrøm As" company as part of improvement scheme for the reliable interpretation of silty clay and other soil types within the municipality. The work comprised of 9 CPTu soundings (500, 501, 502, 503, 504, 505, 506, 507, 508), three rotary percussive boring at sites (503, 505 and 506), soil sampling (sensitivity, plasticity index and water content) and pore pressure measurements (at locations of 500, 502, 506, 507) and CAUC-triaxial tests (at site502) refer Appnedix-1 for details. Drillings were used as a means of comparison with CPTu data and confirming the general stratigraphy of the site together with some 54mm diameter samples for laboratory analysis. This study was performed mainly to investigate the mechanical properties of silty clay based on zone 502, Gunnestad in Sande municipality. Result of new laboratory samples with the collected in situ CPTu raw data are the subject of this study.

Table 1 Location coordinates of 9 CPTu soundings in Sande municipality

Number	North	East	Height
500	6607120.3497	568510.6578	19.5645
501	6606318.1321	568277.5520	14.6933
502	6606857.3685	568476.2248	17.8272
503	6606280.5317	568380.0326	12.8319
504	6606080.1432	568364.1461	13.1300
505	6605860.5270	568320.4947	17.5391
506	6605899.4505	568344.4368	11.5185
507	6605806.0235	568573.3359	8.2326
508	6605802.8891	568244.7494	10.5267

1.5 Organization of Thesis

This thesis comprises five chapters. The first chapter presents the problem statement and research objectives. In Chapter 2, literature on soil classifications, overconsolidation ratio, mechanical properties of silty and clay soils are reviewed. Chapter 3 deals with systematic approaches on how to solve the problem. The material and the test procedures used are explained. In chapter 4, the results of the laboratory tests and field data are presented and analyzed. Results are discussed and compared to each other and as well as with previous works in chapter 5. Conclusions and recommendations are given in chapter 6.

2. LITERATURE REVIEW

This chapter provides a review of important literature mainly concerned to the previous study which will be used as a background for this particular study. This chapter includes factors affecting CPTu measurements, methods of interpretation for soil classification and overconsolidation ratio. It also incorporates a review of significant elements that can describe the mechanical properties of silty and clay soils which can help to determine the mechanical properties of silty clay which is the target of the study.

2.1 Correction for pore pressure effects

Cone resistance (q_c) and the sleeve friction (f_s) values which are measured from cone penetration test have to be corrected for pore pressure effects in order to have a tangible data for interpretation (Baligh, et al. 1981; Campanella, et al. 1982).

Pore pressure measurement can be affected by three aspects of cone design such as Pore pressure element location, Unequal end area effects, Saturation of pore pressure measuring system (Robertson, and Campanella, 1983). Correcting the cone resistance, consistent results can be obtained to comparing different internal geometry of 10cm^2 cones. It is the pore water pressure which is generated as a result of cone penetration in the ground affects the cone resistance and the sleeve friction. This is mainly due to the inner geometry of a cone penetrometer the ambient pore water pressure (u_T) that acts behind the cone and on the ends of the sleeve friction, see fig-2. The cross sectional area of the load cell (A_n) differs from the projected area of the cone A_c (Lunne et al., 1997; Powell and Lunne, 2005; Campanella et al. 1982). The unequal area ratio “a” called effective area ratio of cone which is A_n / A_c affects the total stress which has to be determined from the cone and the sleeve (Mayne, 1991; Powell and Lunne, 2005).

A cone area ratio as low as 0.38 is considered unacceptable especially in a very soft fine grained soils. Many cone penetrometers have the value of cone area ratio of 0.9 to 0.55 and ideally should be close to 1(Powell and Lunne, 2005). Senneset et al., (1988) suggest that filter position and degree of saturation are also mentioned as additional effects. For example, the use of CPTu, pore pressure measurements for onshore testing do not give reliable results mainly due to loss of saturation of the pore pressure element. But in offshore geotechnical practices, pore pressure measurements by CPTu are repeatable and reliable as there is high ambient water pressure that confirms saturation (Robertson, 2009).

The corrected cone resistance (q_t) and corrected sleeve friction (f_t) is given as follows in equation-1 and equation-2. Importance of cone resistance is important as uncorrected q_c may account for some variations in cone factor which determine the undrained shear strength (Robertson, and Campanella, 1983).

$$q_t = q_c + (1-a).u_T \qquad \text{Eq-1}$$

$$f_t = f_s + (1-b).u_T \qquad \text{Eq-2}$$

Nowadays, most of commercial cones have nearly equal end area friction sleeve that helps to ignore the need for any correction to f_s and provide reliable sleeve friction values. But unequal area effect prevail to some extent that necessitates correction of the cone resistance q_c to the corrected total cone resistance, q_t.

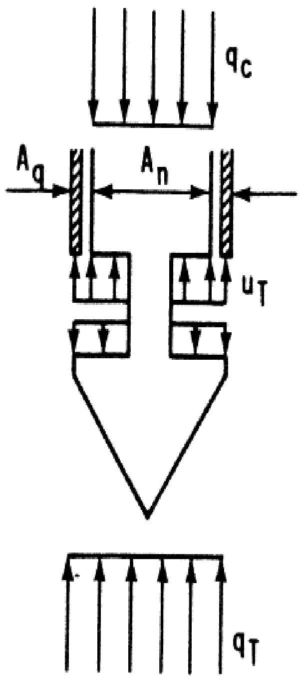


Figure 2 Correction of CPTU recording for Pore pressure effects (Powell and Lunne 2005).

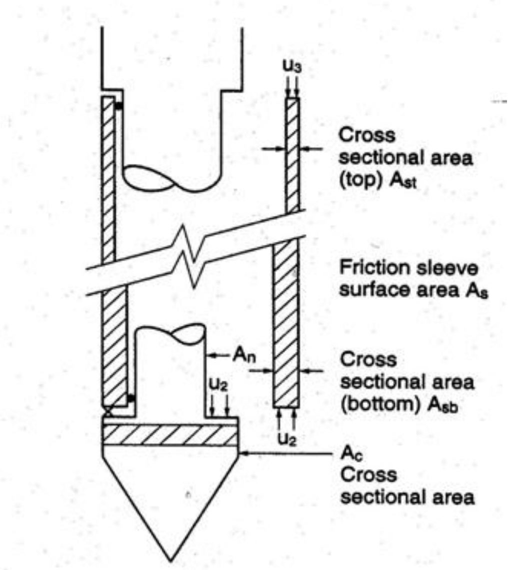


Figure 3 Section through piezocone showing pore water pressure effects on measured parameters (Senneset et al., 1988).

According to Fig 3, the excess pore pressure measured at (upper) u_3 and (lower) u_2 ends of the friction sleeve are different. Using those two parameters and the cross sectional area for the top (A_{sb}) and bottom sleeve (A_{st}), f_T is given as:

$$f_T = f_s - ((u_2 . A_{sb} - u_3 . A_{st})/A_s) \qquad \text{Eq-3}$$

It is mentioned that magnitude of correction can be minimized provided A_{sb} (bottom sleeve cross sectional area) and A_{st} (top sleeve cross sectional area) are equal and making the end areas as small as possible (Powell and Lunne, 2005). Mayne and Bachus, (1988) also explained that the position of the porous element for measurement of pore water pressure has not yet been standardized but mostly piezocone can be separated into type1 where pore pressure is measured on the tip of the cone, and type 2 where measurement of pore pressure is taken behind the cone. And it is now common that cone pressures are measured behind the cone, u_2 position. Even though pore pressure measurements is not much reliable than the cone resistance, it is necessary to use for correction to q_t in soft fine grained soils, to conduct dissipation test and also helps to evaluate drainage conditions and soil behaviour type (Robertson, 2009). Due to variation of stresses and strain around a cone, pore pressure measured at the cone face tends to be higher than the one measured behind the tip (Robertson, and Campanella, 1983).

2.2 soil classification

2.2.1 Introduction

It has been over 40 years that cone penetration test (CPT) to be used as a method of field site investigation and replaced the traditional methods such as drilling and sampling due to its fastness, ability for continuous measurements, strong theoretical background, repeatability and economical behaviour of cone penetration test (Tumay et al. 2008; Robertson 2010). Identification of soil type and determination of stratigraphy are one of the main applications of CPT and is made by relating cone parameters to type of soil.

A number of CPT soil profiling methods by (Begemann, Schmertmann, Sanglerat, Searle, Douglas and Olsen, Vos, Olsen and Mitchell) was developed before the advent of piezocones as a result they do not give pore pressure measurements that helps to correct q_c. So first improvement has been made by Robertson et al., (1986) and Campanella and Robertson, (1988) chart based on q_t and R_f. The uncorrected cone resistance (q_c) can lead to a large error in fine grained soils but for course grained soils due to small difference between q_c and q_t both can be used equally well for soil identification (Robertson 2010).

The soil chart made by Robertson et al., (1986), identifies 12 soil types as can be seen in Table-2. The special feature of this profiling chart is due to addition of zone 1(Sensitive fine grained soil), zone 11(Very stiff fine-grained soil) and zone 12 (Overconsolidated or cemented sand to clayey sand) which enables the CPTu to define and delineate all soil behaviour type. This type of chart can work well up to a depth of 30m. But normalized parameters are essential to neutralize for the cone resistance dependency on the overburden stress that would be applied also well to deep CPTu soundings at different sites (Robertson 2009; Robertson and Cabal, 2010). Two new CPTu profiling methods such as Eslami and Fellenius profiling 1997 chart and Robertson 1990 chart) are from the developed charts where their classifications gives accurate soil type determination and have been tested in sands, normally consolidated clays and overconsolidated clays (Eslami and Fellenius, 2004). However soil classifications with different approaches, can be viewed as diagnostic tool to provide realistic classification (Cai, et al. 2011).

2.2.2 Robertson soil classification

2.2.2.1 Robertson 1990 method:

Robertson 1990 method nowadays is one of the well known soils behavioural classification charts for post processing results using normalized piezocone parameters (Schneider, et al. 2008). Robertson 1990 modified the previous soil classification chart made by Robertson et al., 1986 by plotting normalized cone resistance against normalized friction ratio in a cone resistance chart. While the pore pressure ratio chart uses normalized cone resistance against pore pressure ratio, B_q uses the same limits as previous chart, Fig-4 (Lunne et al. 1997).

Their soil behaviour type for SBT and SBTn chart is mentioned in table-2. According to Robertson 2009, the chart based on Q_t - F_r is recommended than Q_t - B_q chart as it provides the best overall success rates for soil behaviour type (SBT) compared with samples. Besides, Q_t - F_r chart is widely used in onshore geotechnical practice than Q_t - B_q chart as there is lack of pore pressure readings above the water table and its difficulties in maintaining saturation when passing through partially saturated material or in stiff and dilatant deposits (Schneider, et al. 2008).

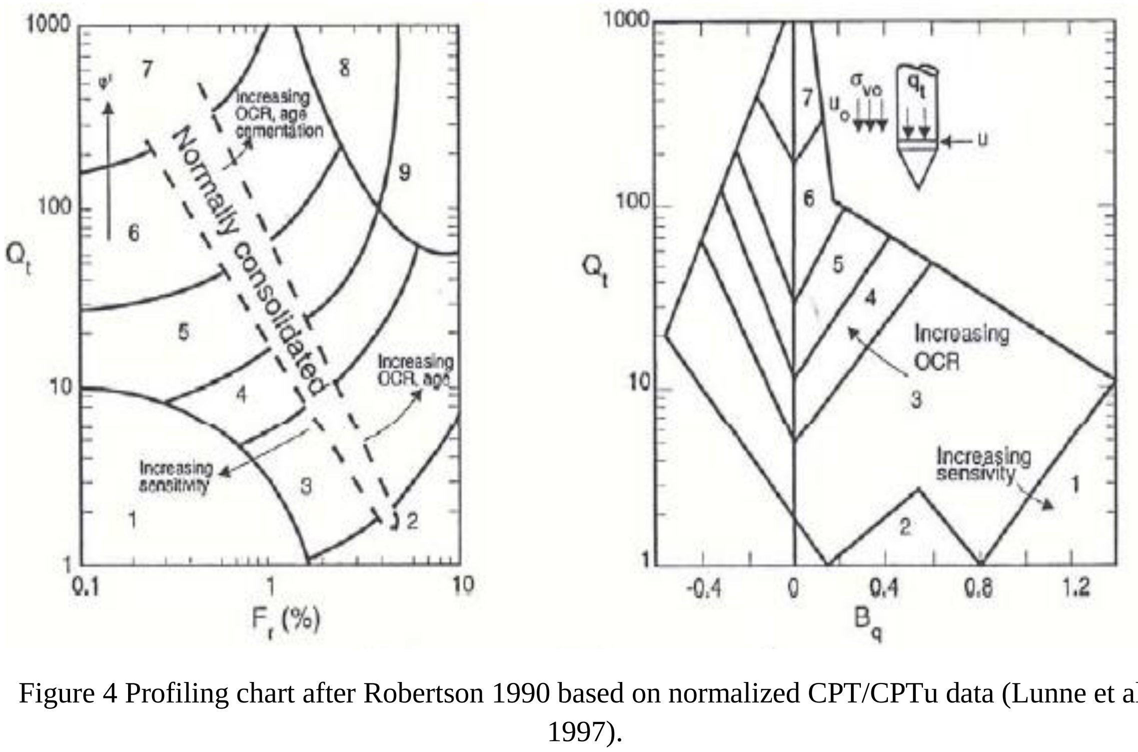


Figure 4 Profiling chart after Robertson 1990 based on normalized CPT/CPTu data (Lunne et al. 1997).

The projected normalized cone parameters of Robertson 1990 are as follows:

$$Q_t = (q_t-\sigma_{vo})/ \sigma'_{vo} \qquad \text{Eq-4}$$

$B_q = (u_2 - u_o)/(q_t - \sigma_{vo})$ Eq-5

$F_r = F_s/(q_t - \sigma_{vo})$ Eq-6

Despite its popularity and wide use, sometimes, soils like organic silts may fall in different zones within the chart that should require personal judgment and relating with borehole information (Hazma et al. 2005). In addition to this, soil classification criteria based on grain size distribution and plasticity can fit reasonably well to soil behaviour type obtained from the field. But even though there is good agreement between unified soil classification system (USCS) and CPT-based soil behaviour type, but differences can arise in the presence of transitional soil types or mixed soils. So to avoid this misinterpretation in transitional soil types (in transition at or near an interface between soils having different soil strength and stiffness), Robertson 1990 chart is modified by adding SBTn index, I_c which will be further explained down below (Robertson, 2009).

Table 2 Common description of soil behaviour type for Robertson 1986 method (Robertson et al, 1986) and Robertson, 1990 method (Robertson, 2010) together with Ic values (Robertson and Cabal, 2010)

SBT zone (Robertson et al., 1986)	**SBT_n zone (Robertson et al., 1990)**	**Proposed common SBT Description**	**I_c**
1	1	Sensitive fine-grained	N/A
2	2	Clay – organic soil	3.6
3	3	Clays: clay to silty clay	2.95- 3.6
4&5	4	Silt mixtures: clayey silt & silty clay	2.6 – 2.95
6&7	5	Sand mixtures: silty sand to sandy silt	2.05 – 2.6
8	6	Sands: clean sands to silty sands	1.31 – 2.05
9&10	7	Dense sand to gravely sand	< 1.31
12	8	Stiff sand to clayey sand*	N/A
11	9	Stiff fine-grained*	N/A

*overconsolidated or cemented, N/A=not available

2.2.2.2 Modified Robertson 1990 method

An SBT type index, I_c was first identified by Jefferies and Davies (1993) that this parameter can represent the SBT in a normalized chart. With a combination of Robertson 1990 (the normalized cone parameters Qt and Fr) and Soil Behaviour Type index, I_c, it has now possible to identify the transition from one soil type to another. A number of studies have demonstrates that the normalized SBT_N chart as can be shown in Figure-5, has greater than 80% reliability when compared with samples. Where I_c is the radius of the essentially concentric circles that represent the boundaries between each SBT zone, refer eq-7. In addition I_c value does not apply for zones 1, 8 and 9 as can be seen in table-2 (Robertson and Cabal, 2010). I_c can be defined as follows;

$$I_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5} \qquad \text{Eq-7}$$

Where Q_t = normalized cone resistance and F_r = friction ratio.

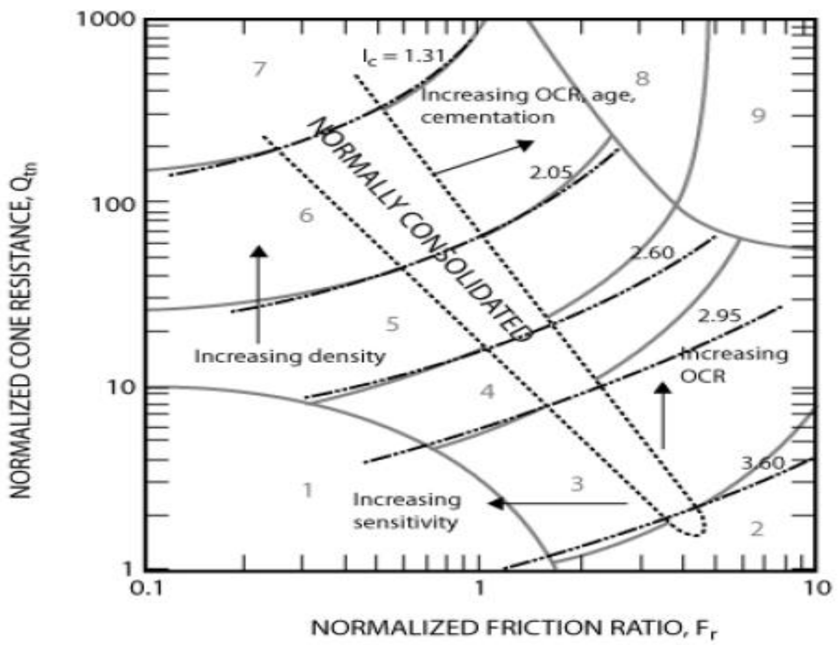


Figure 5 Updated normalized SBT chart based on normalized SBT (Qt-Rf) dimensionless cone resistance, showing contours of Ic (Robertson and Cabal, 2010).

2.3 Stress History- Overconsolidation Ratio (OCR)

2.3.1 General Description and Background

Over consolidation ratio (OCR) are typically defined as the ratio of the maximum past effective consolidation stress (σ'p) and the present effective overburden stress, σ'_{vo} (Lunne et al., 1997).

$$OCR = \sigma'p / \sigma'_{vo} \qquad \text{Eq-8}$$

For mechanically over consolidated soils where the only change has been the removal of overburden stress, the above ratio is appropriate. However, for cemented and/or aged soils the OCR can be represented by the ratio of the yield stress and the present effective overburden stress. The direction and type of loading affects the value of yield stress (Robertson and Cabal, 2010).

Studying stress history is very crucial to observe their effect on mechanical properties of sediments (Tonni and Gottardi, 2011). With the advent of piezocones being as one of the important tools for profiling the over consolidation ratio (OCR= σ_p'/ σ'_{vo}) in clay deposits, many researches were done to establish a correlation (based on the cone resistance, friction sleeve and pore water pressure) by establishing a number of theoretical and empirical parameters with OCR. Even though most of the empirical and theoretical parameters give good correlation with OCR, most of them are site specific and does not apply well to soft soils of different origin and different geographical location (Mayne and Bachus, 1988).

2.3.2 Methods for Estimation Stress History

Three methods such as indirect correlation via undrained shear strength (s_u), methods based on the shape of the CPTu profile, direct correlation with CPTu data have been mentioned by (Lunne et al., 1997) to estimate OCR from CPTU/CPT soundings. But for this research direct interpretation from CPTu data and specifically estimation of OCR by Karlsrud et al., (2005) was the main concern where sensitivity of soils was taken into consideration.

2.3.2.1 Direct Interpretation Using CPTU Data

It has found that the ratio of total measured pore pressure to cone resistance (u_{max}/q_c) are inversely proportional with the OCR which shows sensitivity of stress history with pore pressure .This later followed by a number of relationships between OCR and various forms of normalized pore pressure and normalized cone resistance values. The most important ones are shown in table-3. It is described that there are no unique relationship that exists between OCR and pore pressure due to the factors such as clay sensitivity, pre-consolidation mechanism, soil type and local heterogeneity (Mayne and Holtz, 1988). The pore pressure parameter B_q = $\Delta u/q_c$- σ_{vo} is used as one of the alternatives in estimating OCR. But it shows a wide range of OCRs for specific values of B_q. besides, it is site specific and demands detailed calibration with high quality samples and oedometer test (Mayne and Holtz, 1988; Mayne, 1991; Been, et al. 1993). Karlsrud et al. (2005) also described that there is high scatter of OCR which is based on B_q values. To some extent better correlation can also be found based on excess pore pressure (Δu). However, best correlation is found on the basis Q_t as shown in the equations-9 and 10 but it is recommended to use all correlation to determine the undrained shear strength on specific projects.

For low sensitive clays (S_t< 15)

$$OCR = (Q_t/3)^{1.20} \qquad \text{Eq-9}$$

For high sensitive clays (S_t> 15)

$$OCR = (Q_t/2)^{1.11} \qquad \text{Eq-10}$$

Table 3 List of Piezocone parameters for estimating OCR (Lunne et al., 1997).

Parameter	**Basis**	**Reference**
U_m/q_c	empirical	Baligh et., 1980
$\Delta u/q_c$	empirical	Campanella & Robertson, 1981
$B_q= \Delta u/(q_c-\sigma_{vo})$	empirical	Senneset, Janbu, & Svanø, 1982
$B_q= \Delta u/(q_t-\sigma_{vo})$	empirical	Wroth, 1984
$\Delta u/(q_c-u_o)$	empirical	Smith, 1982
$\Delta u/\sigma'_{vo}$	empirical	Azzouz et al., 1983
$\Delta u/\sigma'_{vo}$	Theory	Mayne & Bachus, 1988
$N_u= \Delta u/s_u$	empirical	Tavenas & Leroueil, 1987
$q_t-\sigma_{vo}$	empirical	Tavenas & Leroueil, 1987*
q_t-u_m	Theory	Konrad & Law, 1987*
Δu	empirical	Mayne & Holtz, 1988*
q_t-u_0	Theory	Sandven, Senneset & Janbu, 1988*
$(q_t-\sigma_{vo})/\sigma'_{vo})$	Theory	Wroth, 1988
$(u_1/u_0)-(u_2/u_0)$	empirical	Sully et al., 1988
q_t,u_1,f_t	empirical	Rad & Lunne, 1988
$(q_t-u_2)/\sigma'_{vo}$	Theory	Houlsby, 1988 and Mayne 1991
$F_t/(q_t-\sigma_0)$	empirical	Wroth 1984

Other approaches have been formulated such as combining the theories of cavity expansion and critical state soil mechanics to form a reasonable expression for OCR in terms of normalized excess pore water pressure ($\Delta u/\ \sigma_{vo}$'). This method has been previously recommended for very soft clays and was tried for piezocone soundings in stiff sandy marine clays and the stress history at a specific site is fairly established. The approach was further tried in 32 different sites and its applicability subsequently substantiated (Mayne and Bachus, 1988). Based from cavity expansion theory, Kulhawy and Mayne, (1990) suggested the following expression:

$$OCR = k(q_t-\sigma_{vo})/\sigma_{vo}' = kQ_t \quad \text{or} \quad \sigma'_p = k(q_t-\sigma_{vo}) \qquad \text{Eq-11}$$

Where expected range of k is from 0.2 to 0.5 with an average value of k = 0.33. Higher values of k are recommended for in aged and heavily overconsolidated clay. But this form is valid for $Q_t < 20$.

An approximate forms of the model relate OCR directly to $(q_t-\sigma_{vo})/\sigma_{vo}'$ was tested to a database for all clay types that includes soft to stiff to hard intact and fissured clays and verify the approach. The value of k is 0.81 if pore pressure is measured mid face element, u_1 and 0.46 if pore pressure is measured behind the cone, u_2. It is also mentioned that interesting and similar outcome was found from other researchers (Chen and Mayne, 1996).

The estimation of stress history; the correlation parameter $k(q_t-\sigma_{vo})/\sigma_{vo}'$ is most important one if little experience is available. Other parameters such as $B_q = \Delta u/(q_t-\sigma_{vo})$, $\Delta u/\sigma'_{vo}$, and $f_t/(q_t-\sigma_{vo})$ is also recommended to be applied to estimate OCR based on the conservative average of consistent data. The normalized excess pore pressure ($\Delta u/\sigma'_{vo}$) shows different pore pressure results for normally consolidated and overconsolidated clays and can help as an indicator of stress history (Mayne, 1986; Campanella and Robertson, 1988). But for lager projects additional high quality laboratory and field data is necessary with site specific correlations which depend on consistent values of OCR (Lunne et al., 1997). Oedometer test with constant rate of strain (CRS) can estimate the preconsolidation stress (P'_c) at the rate of strain similar to the undrained strength (Karlsrud et al., 2005). The preconsolidation stress (P'_c) can also be estimated from uniaxial test using the old but reliable Casagrande's method with the help of log plot graph. This method is commonly used to find P'_c value (Dawidowski and Koolen 1994).

2.4 Mechanical properties of silty and clay soils

2.4.1 Effects of sensitivity and plasticity index

Sensitivity (S_t) can be defined as the ratio of undisturbed undrained shear strength to totally remoulded undrained shear strength and it can also be interpreted with the use of sleeve friction (f_s) data from CPT where f_s represent equally to remoulded undrained shear strength (Robertson and Cabal, 2010) . The relationship can be given in percent as shown in Eq-12. Where: N_s is a constant and R_f is friction ratio. According to Rad and Lunne, (1986), the value of N_s an average value of 7.5 is selected from the range of 5 to 10 using the non-normalized friction ratio (R_f). N_s value of 7.3 and 7.1 was suggested by Mayne, (2007) and Robertson, (2009) respectively.

$$S_t = N_s/R_f \qquad \text{Eq-12}$$

Karlsrud et al. (2005) mentioned that sensitivity is one of the main factors that affect excess pore pressure and $N_{\Delta u}$, where experiments were done with the use of high quality samples and CPTu data. Data were divided into two parts (with sensitivity greater and less than 15) and their trend shows inverse relationship for different ranges of OCR values. It is also worth to mention that plasticity index (PI) which is the difference between liquid limit (LL) and plastic limit (PL) can influence cone factor where PI varies with the amount of clay content. Lunne et al. (1997) described that the low values of sleeve friction of relatively high sensitive clays can affect the estimation of sensitivity using CPT data. In addition to this, Sample quality can be affected by clay sensitivity in addition to low plasticity and increasing sample depth which may give wrong interpretation (Karlsrud et al., 1997).

2.4.2 Undrained shear strength of clay

Undrained shear strength can be analysed either from laboratory (e.g. CAUC triaxial test) or based from CPTu/CPT. As it has been mentioned earlier due to fully undrained system it is easy to compute s_u for clay soils based on cone penetration data.

Because of variation of s_u with difference in stress history, soil anisotropy, strain rate and mode of failure, analyzing the undrained strength is mainly dependent on nature of the problem (Sandven and Black, 2004; Hazma et al. 2005; Tong, et al., 2011). As mentioned by Lunne et al. (1997), two methods (theoretical and empirical approach) are applied to determine s_u based on CPTu/CPT data. But for this type of research more credit is given to empirical approach.

Empirical approach:

CPTu/CPT based for determining undrained shear strength can be classified into three main categories such as estimation using: 1. Net cone resistance, 2. Effective cone resistance and 3. Excess pore pressure (Karlsrud, et al. 1997; Tong, et.al., 2011)

Their expression can be given as follows:

$$S_u = (q_t - \sigma_{vo})/N_{kt} \qquad \text{Eq-13}$$

$$S_u = (q_t - u_2)/N_{ke} = q_e/ N_{ke} \qquad \text{Eq-14}$$

$$S_u = (\Delta u)/N_{\Delta u} \qquad \text{Eq-15}$$

Where:

($\Delta u = u_2 - u_0$) = excess pore pressure

σ_{vo} = total in situ vertical stress

q_e = effective cone resistance

N_{ke}, N_{kt}, $N_{\Delta u}$ = cone factors based on effective cone resistance, corrected cone resistance and excess pore pressure respectively.

The undrained shear strength which was analyzed from triaxial test and field vane test can be used as a reference to estimate the cone factor values (Tong et al., 2011). For example Parkin and Lunne, (1982) mentioned that derivation of shear strength; stress history and moduli values are based on semi-empirical correlation through soil sampling and laboratory tests. Difficulties are always faced to obtain good sample from 54mm piston sampling especially in silty marine clays of Norway with water content less than 30% to 40% and plasticity index less than 15% to 20%. So it is recommended to use relatively high quality block samples (Karlsrud et al. 1997). Anisotropically consolidated triaxial test sheared in compression (CAUC) are commonly used by NGI to determine undrained shear strength (Lunne et al., 1985).

It is not advisable to use effective cone resistance (q_e), to determine the s_u for soft normally consolidated clays and heavily overconsolidated deposits (Tong, et al., 2011). This is mainly contributed by sensitivity and small value of q_e due to small error in q_c and u_2 measurements for the above mentioned clay type (Powel, et al., 1988). Karlsrud et al., 1996 as cited in Lunne, et al., 1997, tried correlating B_q and N_{ke} (Fig-6) based on reference s_u which is found from high quality block samples and this works well for normally to lightly overconsolidated clays. Nevertheless, in the case of heavily overconsolidated clay deposits, the negative and small value of B_q can affects the s_u.

Good correlation can be obtained between N_{kt} and B_q with the value of N_{kt} from 6-15 as can be shown in Fig-6. N_{kt} ranges from 10-15 for normally consolidated clays and from 15-19 for overconsolidated clays and use of local correlation is suggested to minimize scattering. The scattering is mainly depends:

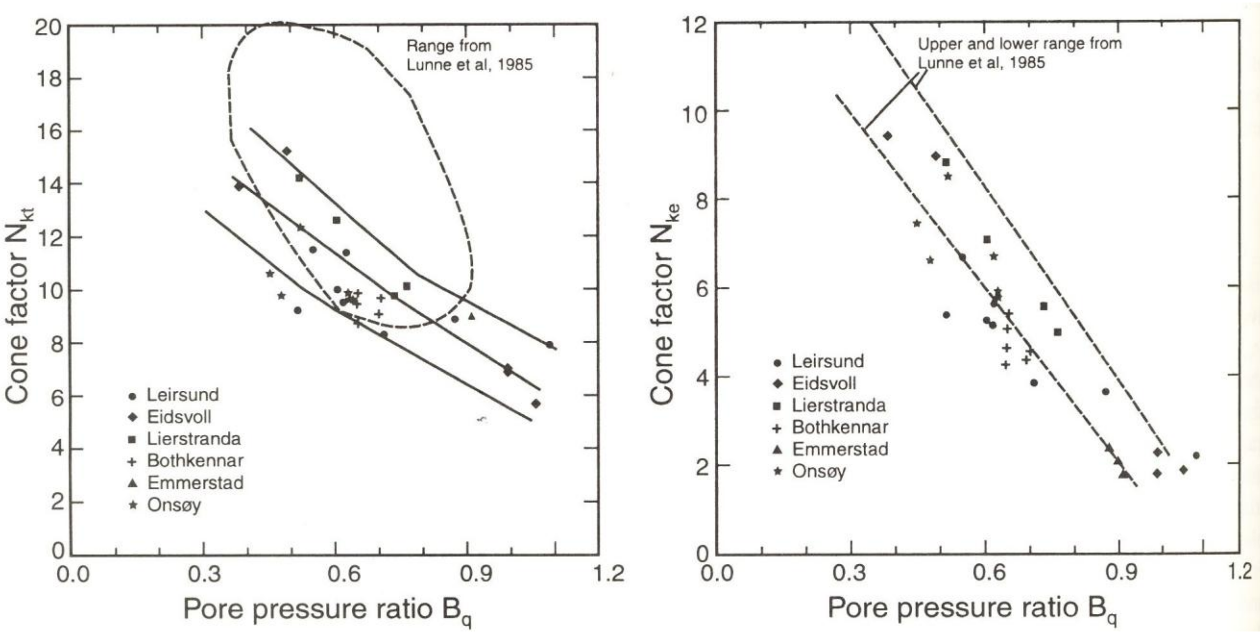


Figure 6 Pore pressure ratio, Bq vs Nkt and Nke (source from Lunne et al. 1997).

1. Due to the absence of unique measurement for the undrained shear strength of soil (Schneider, et al. 2008).
2. The value can depend on the type of test performed (Tong, et al., 2011)
3. The variation in strain rate can affect the value of undrained shear strength (Karlsrud et al., 1997)
4. Different empirical methods can have different undrained shear strength (Senneset et al. 1989).

Lunne et al. (1985) proved for the decreased tendency of N_{kt} with the increase in OCR. However, OCR does not alone control the cone factor which is the reason for poor correlation. So additional factors such as clay type and how the apparent OCR has been developed is also crucial. It is preferred to use B_q and Q_t to observe the variation of cone factor for new sites where OCR correlation may not be reliable. B_q is relatively reliable parameter than Q_t. Using all three cone factors and take a reasonable average to have reliable s_u is one of the good options. (Karlsrud et al., 1997). Karlsrud et al. (2005) on the other hand mentioned that it is advisable to take all the three cone factors to determine the undrained shear strength but attention must be given to $N_{\Delta u}$ where comparison among them can indicate the inconsistency with the individual CPTu tests.

Based from CPT/CPTu data,a range of cone factor, N_{kt} , from 15-20 is used for fine grained soil formations where there is little experience and upper limit is chosen for more conservative estimation. In case of stiff fissured clays it can reach as high as 30 while for normally and lightly overconsolidated clays can be as low as 10. Where there is previous experience available in the same formation, the above mentioned should be adjusted (Lunne et al., 1997). For big projects determining the cone factor depends on site specific correlations where high quality can be collected from laboratory and field (Mayne and Kemper, 1988).

Karlsrud et al. (2005) illustrated that $N_{\Delta u}$ is strongly influenced by OCR, sensitivity and plasticity index. $N_{\Delta u}$ can be determined based from OCR and plasticity index for the two ranges of sensitivity as shown in equations 23 and24. However, it requires good sample quality to estimate correctly.

For low sensitivity clays (S_t< 15)

$$N_{\Delta u} = 6.9 - 4.01\log.OCR+0.07(I_p) \text{ where } I_p \text{ is in \%} \qquad \text{Eq-16}$$

For high sensitivity clays (S_t> 15)

$$N_{\Delta u} = 9.8 - 4.5\log.OCR \text{ where } I_p \text{ is in \%} \qquad \text{Eq-17}$$

$N_{\Delta u}$, obtained from Δu can give accurate measurement of s_u. In very soft clay deposits there might be inaccuracy in the measurement of corrected cone resistance (q_t). So use of excess pore pressure is a good option. The value of cone factor ($N_{\Delta u}$) can range from 7 to 10 and 10 is chosen for more conservative estimate. Δu is computed from pore pressure measured behind the cone (Tong, et al., 2011). Besides, the Stress History and Normalized Soil Engineering Properties (SHANSEP) model, which comprises the vertical effective stress and overconsolidation ratio allows in interpreting more reliable values of undrained shear strength from effective stress and OCR conditions (Ladd et. al. 1977). s_u using this model one can define with the following expression:

$S_u = \alpha * \sigma'_{vo} * OCR^m$ Eq-18

Where: $\alpha = s_u / \sigma'_{vo}$ for normally consolidated clay wi th OCR=1. Results based on block samples made by Karlsrud et al. (2005) express SHANSEP parameters that can fit for clays can be shown in fig-7.

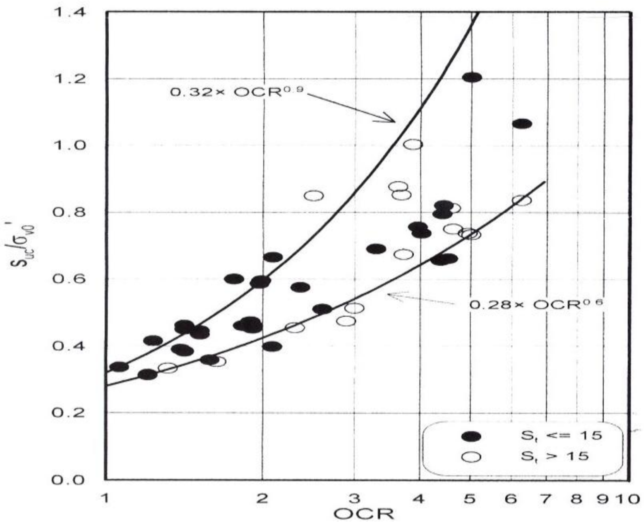


Figure 7 Normalized CAUC su/ σ'vo for block samples in relation to OCR (Karlsrud et al., 2005).

2.4.3 Undrained shear strength of silty soils

Measurement of undrained shear strength cannot represent for soils having B_q values less than 0.4 (representing coarser than clayey silt) due to the partial drainage of the material under loading (Senneset et al., 1982; Sandven, 2003). But B_q values between 0.3 and 0.4, cone penetration is mainly treated as undrained silty soils while for those with B_q values above 0.5 are mainly undrained clayey soils (Long, et al. 2010). Based on Larsson, (1997) illustrations, shear strength of silty soils is usually determined as both undrained and drained shear strength. However in reality silts are partially drained and the more reliable way is, to use effective strength parameters together with the evaluation of total stress, possible pore pressure and expected dilatancy effects. Clayey silt and silty clay where it is difficult to determine their properties from in situ tests, triaxial test is performed to determine the drained shear strength parameters. Undrained tests can also be performed to assess their true undrained nature.

Effective stress method is recommended for such soils as an option to define the strength of silty soils. Triaxial test using traditional approach even do not give clear interpretation of undrained shear strength on silts mainly because of their dilating effect of silty soils during shear. Estimating s_u can be done based on ; simple deviatoric stress regardless of strain, shear stress at some limiting strain, pore pressure parameter, reaching Mohr-Coulomb line, peak principle stress ratio (σ'_1/σ'_3) and peak pore pressure. Reliable value of s_u, for silty soils can be obtained with limiting strain of about 2% or at peak pore pressure (Long, et al. 2010).

Undrained shear strength (s_u) can be computed from CPTu data using bearing capacity factors such as corrected cone resistance (N_{kt}), excess pore pressure ($N_{\Delta u}$) or effective cone resistance (N_{ke}) (Lunne et al. 1997). Comparing the three factors (N_{kt}) can give reliable result for determination of (s_u) for silty soils (Long, et al. 2010). Senneset et al. (1982) as cited in Larsson, (1997) supported the use of CPT data for determining undrained shear strength for silty soils but consistent result can be found if the pore pressure parameter, B_q is at least 0.4 for normally consolidated soil. For overconsolidated soils, there is no possible general guideline and individual judgment is necessary. Larsson, (1997) also mentioned that drained shear strength properties (like friction angle) in silt can also be determined under drained condition of CPT tests.

2.4.4 Effective friction angle (φ') and Attraction (a)

For the proper analysis of slope stability, effective stress strength parameters (ϕ'and a) are important. Current practice is to estimate these parameters from triaxial testing. For safety reasons generous safety factor is applied to determine effective strength parameters. This is mainly done for the reason that sampling induced densification could raise the values of strength parameters. To minimize strain and provide sufficient safety factor, a safety factor of 1.3 is applied in practice on tanϕ' (Long et. al., 2010). Senneset et al. (1989) mentioned that strength parameters can be also estimated using Mohor-Coulmb criterion with the expression:

$$\tau f = (\sigma'+a)\tan\phi' \qquad \text{Eq-19}$$

Where:
τf = shear strength
σ' = effective normal stress on the failure plane

Effective friction angle can also be determined from CPTu data and the same approach is applied for silty and clayey soils (Lunne et al., 1997). Børgeson, (1981) pointed out that the friction angle can be strongly influenced by testing techniques and the characterization of failure. Based from CPT/CPTu data, Senneset et al., (1982) and Senneset et al., (1988) have developed determination of strength parameters with a method known as effective stress interpretation method. The bearing capacity formula with the application of effective overburden stress, σ'_{vo}, can be expressed as:

$$q_t - \sigma_{vo} = N_m(\sigma'_{vo} + a) \qquad \text{Eq-20}$$

Where:

N_m = is the cone resistance number and can be interpreted also by the following equation

a = attraction

β = angle of plastification, an idealized geometry of failure zone around the advancing cone.

Table 4 Tentative value of plastification angle, β for different soil types (Senneset et al., 1989)

Soil type	**Tentative Values of β (degrees)**
Dense sands, overconsolidated silts, Plastic clays, low-compressible overconsolidated clays	-20 to -10
Medium sands and silts, sensitive clays, high-compressible clays	-5 to +5
Loose silts, clayey silts	+10 to +20

To compute N_m, the value of attraction (a) is necessary in addition to CPT/CPTu data as shown in equation-26. Therefore attraction value can be obtained from triaxial test (Sandven et al., 1988), from the trend of q_t versus σ'_{vo} diagram (Janbu and Senneset, 1974) or from general experience. Table -5 gives typical values of "a" and "tanϕ' " for different soils based from the previous experiments and experience. Despite its difficulty to assess β both theoretically and experimentally, a range of β values can be given based on properties of soils such as sensitivity, compressibility, plasticity and stress history as shown in Table-4 for various soil types. Senneset et al. (1988), classified silty soils with different β values and Comparison of effective strength parameters analysed from triaxial test results and CPTu results can give best agreement for silty soils if β is used between$+15^0$ to $+20^0$. But inconsistency might occur between the field and laboratory result due to sample disturbance in silty soils. Besides, it is necessary to note that silt content in sand may decrease the strength parameters and on the other hand large silt content on clays could increase the strength parameters of clays. Clay minerals such as montmorillonite and smecitite may decrease the friction values below the results given in table-5 (Senneset et al., 1989).

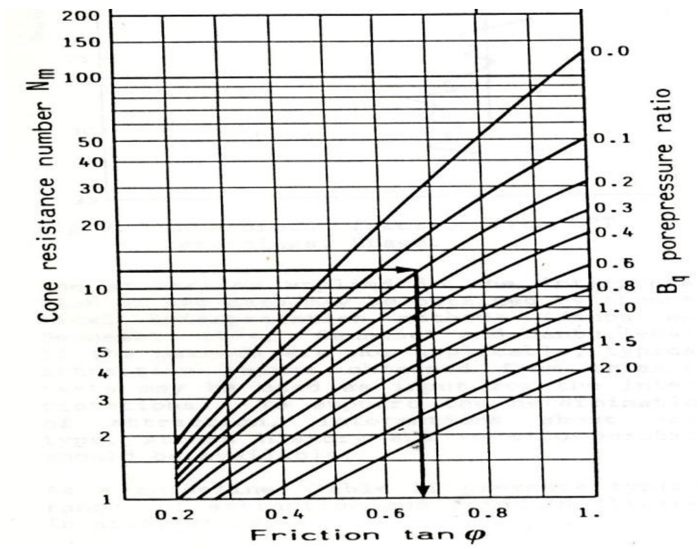


Figure 8 CPT based determination of tanϕ' for β values of 0 (Senneset et al. 1988).

Estimation of friction angle can be obtained with the use of pore pressure parameter (B_q) versus N_m with the chart in Fig-8 as an example with β values of 0^0 but it can range from $+15^0$ to -30^0. Computer based program has been developed for the good interpretation of tanϕ' with the application of B_q versus N_m (Eggereide, K., 1985).

Table 5 Typical values of soil attraction (a) and tanϕ' (Senneset et al. 1989).

	Shear strength parameters				
Type of soil	a(kPa)	tanϕ'	ϕ'(degrees)	N_m	B_q
Clay, soft	5-10	0.35-0.45	19-24	1-3	0.8-1.0
Clay, medium	10-20	0.40-0.55	19-29	3-5	0.6-0.8
Clay, stiff	20-50	0.50-0.60	27-31	5-8	0.3-0.6
Silt, soft	0-5	0.50-0.60	27-31		
Silt, medium	5-15	0.55-0.65	29-33	5-30	0-0.4
Silt, stiff	15-30	0.60-0.70	31-35		
Sand, loose	0	0.55-0.65	29-33		
Sand, medium	10-20	0.60-0.75	31-37	30-100	<0.1
Sand, dense	20-50	0.70-0.90	35-42		
Hard, stiff soil, OC, cemented	>50	0.8-1.0	38-45	100	<0

2.4.5 Deformation and Consolidation characteristics

Determination of deformation parameters from direct cone penetration is difficult as the penetration causes large strains around the soil. Besides the distribution of stresses and pore pressures are not easy to assess. Their result could be different from the real design value and could only be used as rough estimate. Laboratory experiments are of the main concern to have reliable deformation parameters for clays (Senneset et al., 1989).

Based on CPTu data, deformation can be evaluated in the form of constrained modulus, M. Kulhawy and Mayne, (1990) defined general one dimensional constrained modulus for fine grained soils using, $M = 8.25\ (q_t - \sigma_{vo})$. Even though, this correlation applies well; care should be taken in some occasions. Classical general formulation can be given with the following relationship:

$$M = \alpha\ (q_t - \sigma_{vo}) \qquad \text{Eq-21}$$

Where q_t is the corrected cone resistance and σ_{vo} is the total vertical stress. The value of α in the pre consolidation range could vary from 5 to15 for overconsolidated of most clay soils and in the normally consolidated stress range, the value of α can range from 4 to 8. According to Sandven, (2003) and Senneset et al. (1988), the value of α for silty soils can range from 2 to 10. Besides, Janbu, (1985) demonstrates that the initial water content can affect modulus number for clays and silts. M and α can be represented by "M_i and α_i" for overconsolidated clays while for normally consolidated clays it is represented by "M_n and α_n" refer Fig 9. M can also be estimated with the use of Janbu, (1963) relationship as shown in Eq-22. Where M_0 is constrained modulus associated with vertical stress, σ'_{vo} is effective vertical stress, $\Delta\sigma'_v$ vertical stress increment.

$$M = M_0 \sqrt{}\ (\sigma'_{vo} + \Delta\sigma'_v/2)/\ \sigma'_{vo} \qquad \text{Eq-22}$$

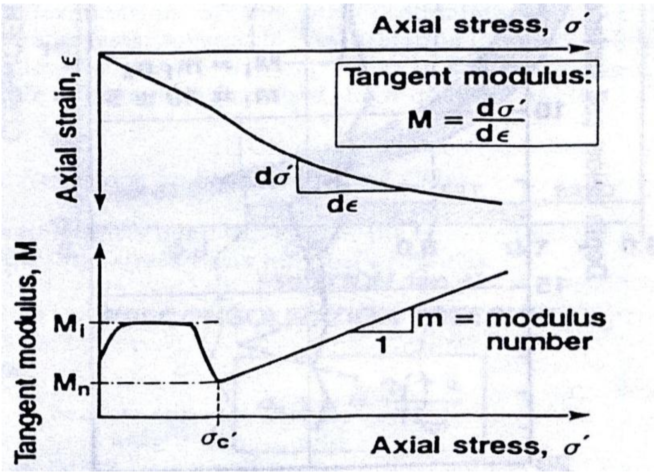


Figure 9 characterization of deformation moduli from CPTU (Senneset et al., 1989).

The stress-strain relationship as can be seen in Fig-9 can work well in uniform soft clays but may not work in silty soils due to the non linearity of the result. But for reliable result site specific correlation based on the value of M from oedometer test is important (Lunne et al. 1997). According to Sandbækken et al. (1987), preconsolidation stress (P'_c), constrained modulus (M), coefficient of consolidation (C_v) and permeability coefficient (k) can be measured from constant rate of strain (CRS) and incremental loading (IL) oedometer tests with good result, provided proper procedure and interpretation is made. The graphic correlation of stress- strain which helps to estimate coefficient of consolidation based from coefficient of permeability and determination of tangent constrained modulus can be shown as in Fig-10 for CRS oedometer test. The test parameters are estimated on the basis of the following expressions listed in table-6.

Table 6 Equations applied for estimating test parameter from oedometer (Sandbæk.ken et al., 1987)

Test parameter	**Formula**
Average effective stress($\sigma'_{a,av}$)	$\sigma'_{a,\,av} = \sigma'_a-(2/3)u_b$
Oedometer modulus (M)	$M = \Delta\sigma'_a/r.\Delta t$
Modulus compressibility (m_v)	$m_v = 1/M$
Coefficient of permeability (k)	$k= ½(r/u_b).H^2.\gamma_w$
Coefficient of consolidation	$C_v = M.k/ \gamma_w$

Where:
r = is the average rate of strain
$\Delta\sigma'_a$ = change in effective stress over an increment of Δt (time interval between readings)
H = height of sample
γ_w = unit weight of water
u_b = excess pore pressure at undrained specimen bottom
σ'_a = total stress at top of specimen

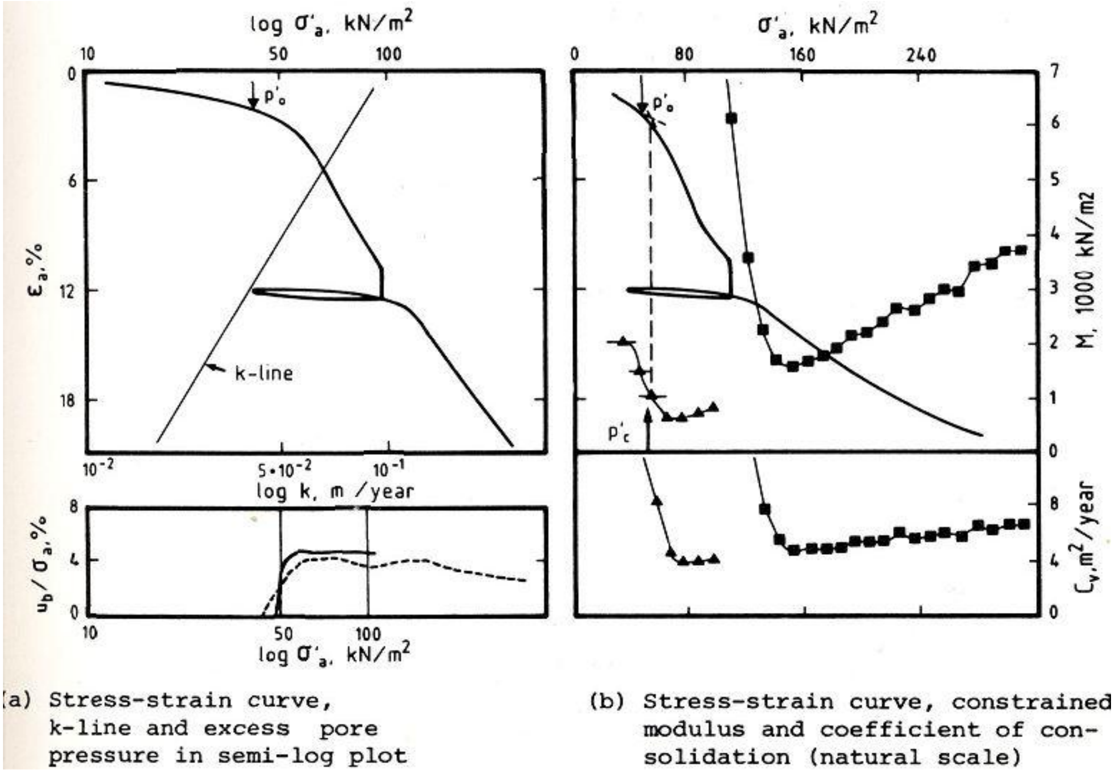


Figure 10 Results of constant rate of strain oedometer test on soft plastic clay (Sandbækken et al., 1987).

3. METHODOLOGY

3.1 Introduction

As it is already mentioned in chapter one, silty clay soils in Sande municipality are partially drained which have the problem in accurate interpretation for evaluation of their mechanical properties which is very crucial in slope stability calculations. This area was previously assessed by different study groups and recommended further research. So the research focused mainly on systematic approach to estimate the strength parameters (ϕ’ and a) and undrained shear strength of silty clay. Besides the deformation and consolidation characteristics were also one of the main interests. This chapter included the research strategy that can give brief understanding of the aim of study. It is then followed by data collection where it includes all the materials used, step by step approaches both for laboratory tests and field data such as CPTu measurements, borehole data and pore pressure measurements. Then data analysis part included every detail of the system that is used to draw meaningful conclusions from the data collected.

3.2 Research strategy

To accomplish the research, 10 CPTu soundings (at 500, 501, 502, 503, 504, 505, 506, 507 and 508 accomplished by GeoStrøm AS and one CPTu sounding at 20061298 from previous study made by NGI), pore pressure measurements (at 500, 502, 506 and 507) three rotary pressure sounding (at 503,505 and 506), four augering (sites 501, 504, 506 and 507) for soil sampling and laboratory tests (for site 502 with more silty soils) were used (refer appendix 1 for detail location). Based on CPTu data different soils at different areas in Sande municipalities were identified and was correlated with some of the available borehole data. In addition, correlating CPTu soundings with overconsolidation ratio and variation of plasticity index and sensitivity with depth were assessed. Piezocone data that evaluates the ground water condition, Liquid limit, plastic limit, water content and sensitivity measurements were taken for analyzing mechanical properties clays and silty sediments. For more silty soils which are partially drained, it is difficult to study and interpret the mechanical behaviour and other properties just same as that of clay using CPTu data. So CAUC type triaxial test and

Oedometer tests were made to estimate the mechanical properties. Fig-12 shows flow chart for strategic plan and framework of data analysis to achieve the objective of the study.

3.3 Data collection

3.3.1 Introduction

Site investigation at different sites, with CPTu data collection, auger drilling for soil identification and sample analysis and pore pressure measurements from piezometers were done by GeoStrøm AS" private company from 13th May, 2011 to 10th November, 2011 in Sande municipality. This was done to have a better image for zone 502. Laboratory tests were applied by NGI for zone 502. One must bear in mind that there might be some errors in collecting the data which might affect the interpretation of this research. Detailed location of the investigation sites is shown in Appendix-1.

3.3.2 Material used and procedures

3.3.2.1 CPTu field data

Measurement of raw piezocone data such as sleeve friction (f_s), uncorrected cone resistance (q_c) and pore pressure measured behind the cone (u_2) were applied by cone penetration tests where a cone on the end of a series of rod is pushed into the ground at a constant rate of 2cm/second. Those parameters helped in identifying the nature and sequence of subsurface strata, ground water condition and physical and mechanical behaviour of soils. The area of the cone used was 10cm^2 with sleeve area of 150cm^2. Before cone penetration applied, predrilling was performed for all cone penetration tests to avoid damaging of the cone. Thrust machine positioned properly so us to attain a thrust direction as near as vertical. The maximum temperature effect at zero load readings was recorded. Area factors "a" and"b" were measured. For details see appendex-2.

3.3.2.2 Piezometer readings

Pore pressure measurements were taken with the use of piezometers at sites close to; 500, 502, 506 and 507 for details one can refer Table-14. In the absence of dissipation tests, this helped for correct interpretation of CPTu soundings (like calculation of vertical effective stress and total effective stress with depth).

3.3.2.3 Borehole information

A rotary pressure sounding is pushed into the ground at a constant penetration rate of 3m/min with a constant speed rotation of 25 RPM. It is performed to map the stratification and locate quick clay deposits. This gives the best indication for the presence of clay content. So data were collected at sites 503, 505 and 506 (Appendix-3). Besides, augering were performed for sites 501, 504, 506 and 507. Samples analyzed for vital information of water content, plasticity index, sensitivity at specific depths with the results shown in Appendex-4.

3.3.2.4 CPT-pro and Macro-Excel data processors:

Interpretation of CPTu measurements was made using CPT-pro software, and Macro-Excel spreadsheet software. The selected CPTu soundings were processed in the CPT-pro software and prepared for further interpretation. At the beginning, corrections of raw data were made such as net cone area ratio, reduction of false measurements and elimination of thin layers. As there were no dissipation test results, pore pressure measurements from piezometers were used for the calculation of in situ pore pressure (u_0). Interpretation of CPT mainly depend on additional auxiliary and derivative parameters, those softwares were capable of calculating:

- Total overburden stress, σ_{vo}
- Effective overburden stress, σ'_{vo}
- Corrected cone resistance, $q_t = q_c + (1\text{-}a)u$
- Corrected local friction, f_t
- Friction ratio, $R_f = f_t/q_t*100\%$
- Normalized cone resistance, $Q_t = (q_t-\sigma_{vo})/ \sigma'_{vo}$
- Normalized friction ratio, $F_r = f_s/(q_t-\sigma_{vo})$
- Pore pressure parameter, $B_q = (u-u_0)/(q_t-\sigma_{vo})$

3.3.3 Laboratory test procedures

Both the CAUC triaxial test and Oedometer test were done by NGI. So the procedures given below are described just to have general information of the test method.

3.3.3.1 CAUC Triaxial tests

Triaxial tests with the device shown in Fig-11 have been one of the special equipment for years and which is used to estimate reliable soil parameters to the maximum possible for an extensive range of geotechnical difficulties and soil types mainly for soft and quick clays. Based on Berre. T. (1981) the brief procedures are:

- A cylindrical soil specimen with 54mm diameter is used. Representative height is made 1 to 1.6 times the diameter and is enclosed in a rubber membrane inside triaxial cell. The test is accompanied by sample preparation, trimming and mounting of the specimen inside the triaxial cell.

- For samples with much silt and sand sample is pushed directly in to a rubber membrane where a suction is made for the sample before the cylinder is detached or removed.

- A filter stone is placed at the top and bottom of the specimen and both are attached to a drainage tubes.

- The specimen is consolidated under compression test called CAUC (anisotropically consolidated undrained compression) and is applied with increasing axial stress and constant radial stress.

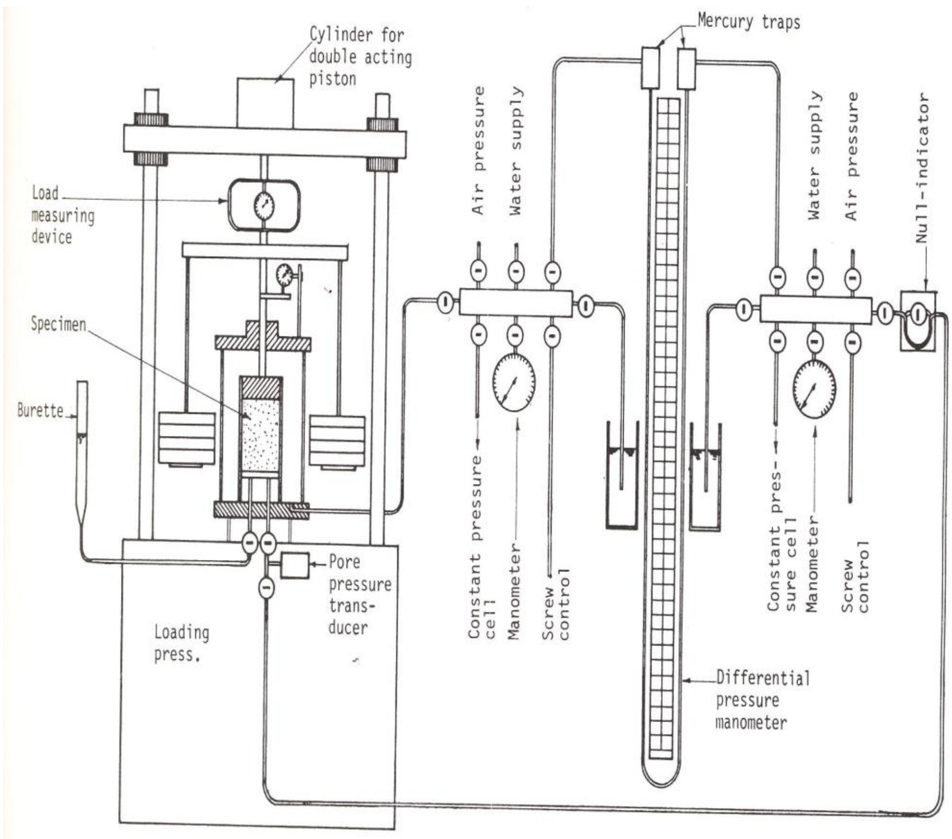


Figure 11 General layout of triaxial testing equipment at NGI (Berre. T., 1981).

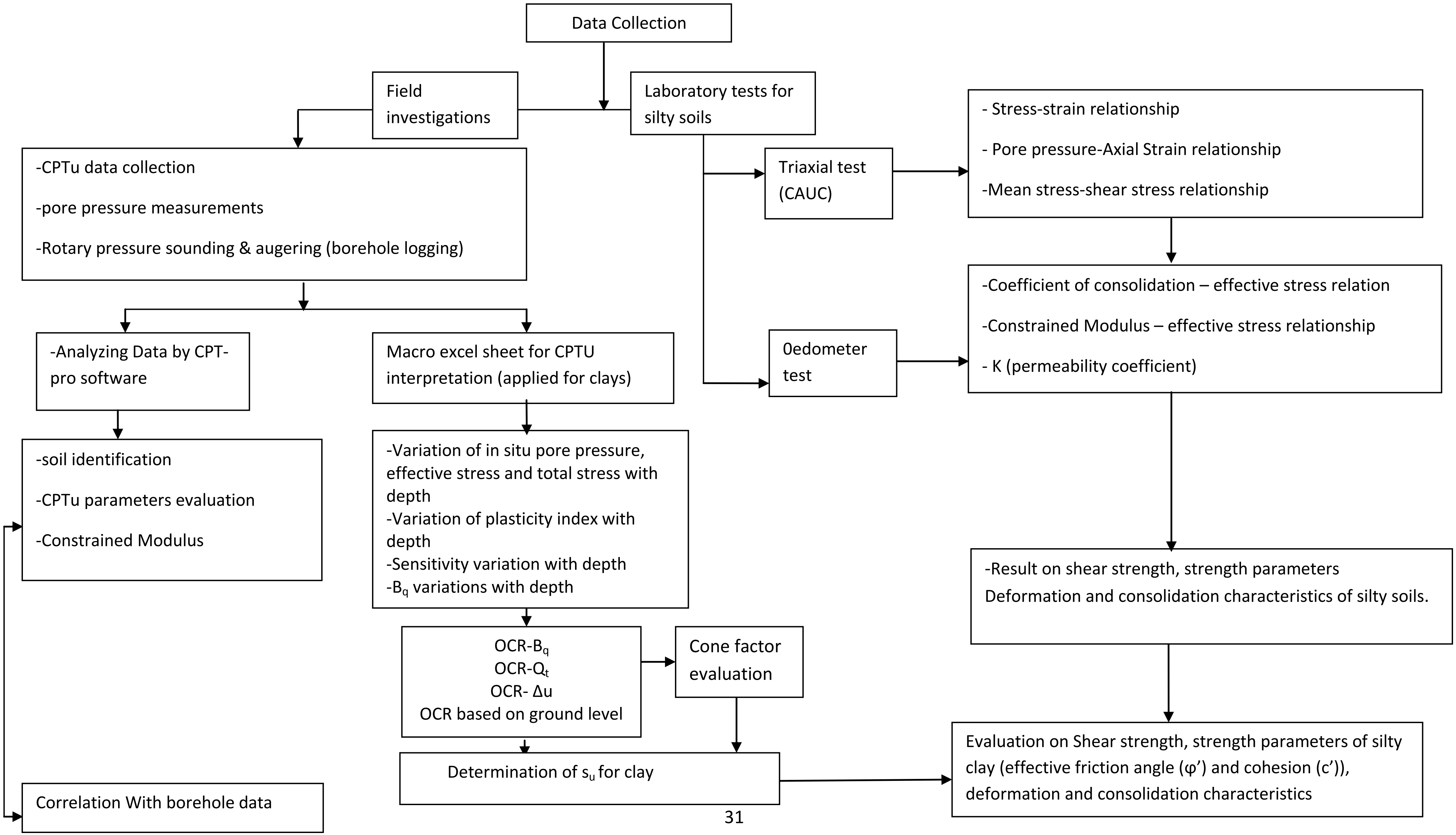

Data Collection
Field investigations
Laboratory tests for silty soils
-CPTu data collection
-pore pressure measurements
-Rotary pressure sounding & augering (borehole logging)
Triaxial test (CAUC)
- Stress-strain relationship
- Pore pressure-Axial Strain relationship
-Mean stress-shear stress relationship
-Analyzing Data by CPT-pro software
Macro excel sheet for CPTU interpretation (applied for clays)
0edometer test
-Coefficient of consolidation – effective stress relation
-Constrained Modulus – effective stress relationship
- K (permeability coefficient)
-soil identification
-CPTu parameters evaluation
-Constrained Modulus
-Variation of in situ pore pressure, effective stress and total stress with depth
-Variation of plasticity index with depth
-Sensitivity variation with depth
-B_q variations with depth
-Result on shear strength, strength parameters Deformation and consolidation characteristics of silty soils.
OCR-B_q
OCR-Q_t
OCR- Δu
OCR based on ground level
Cone factor evaluation
Determination of s_u for clay
Evaluation on Shear strength, strength parameters of silty clay (effective friction angle (φ’) and cohesion (c’)), deformation and consolidation characteristics
Correlation With borehole data

Figure 12 Flow chart for strategic plan and framework of data analysis to achieve the objective of the study.

3.3.3.2 Oedometer Tests

This type of test is very essential in soil mechanics for acquiring parameters in the calculation of consolidation settlements and evaluating stress history. According to NGI for equipment and procedure guidelines, it is possible to get reliable and credible parameters for very soft clays and other difficult parameters. Based on Sandbækken et al. (1987) the steps for performing the test are given in short:

- A cylindrical soil specimen with cross-sectional area of 20, 35 or 50cm^2 and a typical height of 20mm is enclosed in a stainless steel ring as shown in Fig-13.
- Top cap and base plate are supplied with porous stones to which two drainage tubes are linked. Oedometer specimen is always mounted with dry filter stones to avoid swelling of the unloaded specimen.
- Before the continuous loading starts, a stress of nearly one fourth of the in situ vertical effective stress ($1/4p_0$') is applied first. This is followed by an increase in vertical stress at a constant rate of strain.
- Porous stone are flushed with co_2 and become saturated at a state of over burden stress (p_0') and it is continuously loaded to p_1'. This load remains constant from 16 to 24 hours and during this interval, excess pore pressure and vertical displacement are measured.
- Then the specimen is unloaded to p_0', followed by a maximum stress p_2' and permeability measurements are made for this type of test to ensure the back calculated values from the measured pore pressures.
- The coefficient of consolidation is estimated from the coefficient of permeability and the tangent constrained modulus on the stress-stain curve for constant rate of strain (CRS) tests.

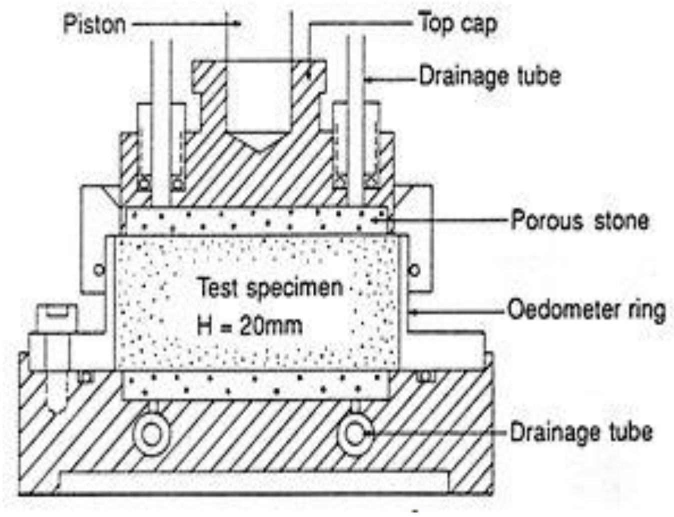


Figure 13 Schematic drawing of 20cm2 oedometer cell (Sandbækken et al., 1987).

3.4 Framework for data analysis

Once the CPTu data are corrected for pressure effects and the in situ pore pressure is taken into account, it is easier to interpret the data and obtain useful output which is helpful to meet the objective of the study together with laboratory tests. This is an important part of the research where it explains in detail all the approaches used to determine the mechanical properties of silty clay.

3.4.1 Soil identification

It has been mentioned in literature study that the best way to identify soil types are Fellenius method, Robertson 1990 method and modified Robertson 1990 method where it accounts soil behaviour type index I_c, which is helpful to identify transition zones in the soil classification chart. However, for this study Robertson 1990 chart (Fig-4) where it correlates normalized CPTu measurements (Q_t vs F_r) were used. For confirmation the computed I_c values were compared with the theoretical I_c values.

This type of chart is global in nature and provides reasonable prediction of soil type but may sometimes show overlap. In addition to I_c, soil samples and borehole data was compared with soil behaviour types to confirm soil behaviour. Robertson 1990 chart (Q_t vs pore pressure ratio, B_q) will not be applied as it is affected in the absence of full saturation.

3.4.2 Factors controlling the undrained shear strength

Determination of undrained shear strength depends on the initial stress state, direction of loading, stress history, degree of fissuring, boundary conditions and other elements. For this research to have a better understanding on their difference in mechanical properties between silty clay and clay, the following factors will be evaluated with the available data.

3.4.2.1 Unit weight and In situ Pore pressure

It is obvious that unit weight differs for different soil types. This is an important parameter together with in situ pore pressure to calculate the effective overburden stress (σ'_{ov}) and total overburden stress (σ_{ov}). Again these two stress parameters are key ones where a number of empirical and theoretical correlations will depend on determining the properties of soil including the undrained shear strength of clays. Once the soils are identified using Robertson 1990 chart method and I_c values then the unit weight for different soil types can be estimated as shown in table-7.

Table 7 Estimation of unit weight on the basis of soil type described in table-2 (Lunne et al., 1997)

zone	1	2	3	4	5	6	7	8	9	10	11	12
Approximate unit Weight (KN/m^3)	17.5	12.5	17.5	18	18	18	18.5	19	19.5	20	20.5	19

Samples from borehole data helped for more confidence to estimate unit weight of soils. As there is no dissipation pore pressure data from piezocone, piezometer readings (Table-14) was used to compute the insitu pore pressure. According Craig, (2004), total vertical stress and effective stress are given by the equations below:

$$\sigma_{ov} = \gamma_{sat} .Z \qquad \text{Eq-23}$$

$$u_0 = \gamma_{water} .Z \qquad \text{Eq-24}$$

$$\sigma'_{ov} = \sigma_{ov} - u_0 \qquad \text{Eq-25}$$

where u_0 is the hydrostatic pore pressure, γ_{sat} = saturated unit weight of soil, γ_{water} = unit weight of water and Z is the depth unit of water is given as 9.8KN/m^3. The correlation of these three parameter in relation with depth is illustrated as shown in Fig-14.

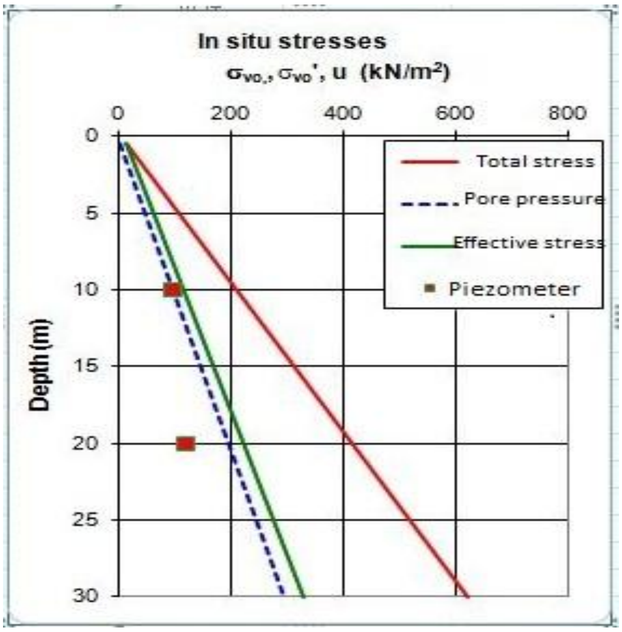


Figure 14 shows variation of σov , σ'ov and u with depth.

3.4.2.2 Plasticity index, Sensitivity and water content

OCR does not alone affect the determination of cone factors. Clay type and content has also a major role in determining accurately the cone factor value. Consideration of plasticity index, sensitivity and the available water content of different clays were an interest of this study. Variation of plasticity index and sensitivity with depth were assessed at different sites.

3.4.2.3 Overconsolidation ratio (OCR)

OCR of clays is one of the main parameters that affected the cone factors which of course the undrained shear strength and deformation characteristics of soils. According to the previous study made (Mayne, 1986; Campanella and Robertson, 1988), the parameter $k(q_t-\sigma_{vo})/\sigma_{vo}'$ is one of the most important correlation parameter to estimate stress history for areas of little experience. Parameters such as $B_q= \Delta u/(q_t-\sigma_{vo})$, $\Delta u/\sigma'_{vo}$, and $f_t/(q_t-\sigma_{vo})$ is also suggested to be applied to estimate OCR based on the conservative average of consistent data. But for this type of research where the aim is to differentiate the mechanical property of silty clay from clay, particular parametric equations should account the variation of clay content where it can be reflected by variation in plasticity index, water content and sensitivity. Correlation derived by Karlsrud et al., (2005) to determine OCR on the basis of Q_t was best method to achieve the desired goal (refer Eq-26 and Eq-29). OCR as a function of B_q and Δu and ground level was used as a means of comparison (Eqs-27, 28, 30 and 31). For consistent site specific correlation, high quality laboratory data from oedometer tests was tried for site 502.

For low sensitive clays (S_t< 15)

$$OCR = (Q_t/3)^{1.20} \qquad \text{Eq-26}$$

$$B_q = 0.88\text{-}0.51.\log OCR \qquad \text{Eq-27 } (S_t =< 15)$$

$$\Delta u = 2.4 + 8\log . OCR \qquad \text{Eq-28 } (S_t =< 15)$$

For high sensitive clays (S_t> 15)

$$OCR = (Q_t/2)^{1.11} \qquad \text{Eq-29}$$

$$B_q = 1.15\text{-}0.67.\log OCR \qquad \text{Eq-30}$$

$$\Delta u = 2.5 + 6\log . OCR \qquad \text{Eq-31}$$

3.4.2.4 Casagrande's method (p_c' determination)

The oedometer test was not able to clearly indicate the point of preconsolidation stress (p_c') like plastic clays as a result this method was applied from consolidation test results. The threshold point was determined from uniaxial compression test with a graphical method in a semi log plot as can be shown in Fig-15 (Dawidowski and Koolen, 1994). The procedures can be illustrated:

- Virgin compression line (I) was determined.
- Point T with smallest radius of curvature was selected from line curve "II".
- From the smallest radius of curvature, a tangent "t" to the curve and a horizontal line "h" was drawn.
- The angle between those two line was bisected
- Then a point of intersection "c" was found where line "c" meet line "I". And this point indicated the preconsolidation stress.

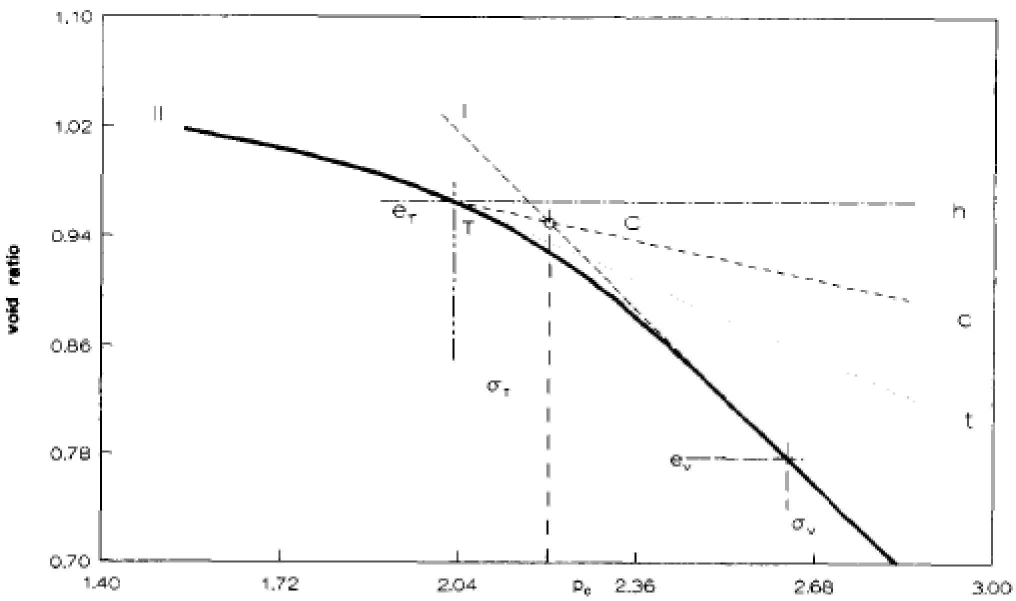


Figure 15 Determination of preconsolidation stress from Casagrade's graphical method (Dawidowski and Koolen 1994).

3.4.3 Undrained shear strength of clay

Unlike the partially drained soils, determination of undrained shear strength of clay is easy to estimate from piezocone data. The cone factor (N_{ke}) was not used for the study due to sensitivity and small value of effective cone resistance (q_e) in normally consolidated soft clays and heavily overconsolidated deposits. $N_{\Delta u}$, determined from Δu was used for the accurate measurement of s_u. N_{kt} was also used as a means of comparison with $N_{\Delta u}$. Determination of s_u using SHANSEP model used as reliable as $N_{\Delta u}$ based s_u especially for clays using equation-18 mentioned in section 2.4.2.

N_{kt} and $N_{\Delta u}$ as a function of sensitivity, plasticity index and OCR applied by Karlsrud et al., (2005) was used for this experiment. The equations applied were:

For low sensitivity clays (S_t< 15)

$$N_{\Delta u} = 6.9 - 4.01\log.OCR + 0.07(I_p) \text{ where } I_p \text{ is in \%} \qquad \text{Eq-32}$$

$$N_{kt} = 7.8 + 2.5\log OCR + 0.0821 I_p \qquad \text{Eq-33}$$

For high sensitivity clays (S_t>15)

$$N_{\Delta u} = 9.8 - 4.5\log.OCR \text{ where } I_p \text{ is in \%} \qquad \text{Eq-34}$$

$$N_{kt} = 8.5 + 2.5\log OCR \qquad \text{Eq-35}$$

Once cone factors are determined, undrained shear strength can be computed by:

$$S_u = (q_t - \sigma_{vo})/N_{kt} \qquad \text{Eq-36}$$

$S_u = (\Delta u)/N_{\Delta u}$ Eq-37

3.4.4 Mechanical properties of silty clay

3.4.4.1 Undrained shear strength and Strength parameters

As silty clay has behaviour on the border between clay and silty soils, it has the probability to be treated either as undrained condition or partially drained. Attention was given on grain size to differentiate soil types. Some of them might have small amount of silt which can be considered the same as clay and others might have higher silt content where it can be interpreted as partially drained silty clay.

According to Schneider et al. (2008); Tonni and Gottardi (2011), soils such as clayey sands and silts, silty clay and silts and many residual soils can be conducted under conditions of partial drainage. More preferable one was to perform laboratory test for silty clay to have reliable result of undrained shear strength. As a result for more silty sediments (which might include silty, clayey silt or silty clay) CAUC triaxial test was performed for three samples at different depth. In this type of test, stress- strain behaviour was assessed as a function of shear stress $(\sigma'_a - \sigma'_r)/2$ vs axial strain (ε_a). In addition to this, the response of (pore pressure as a function of strain rate) and (mean stress $((\sigma'_a + \sigma'_r)/2)$ vs shear stress $((\sigma'_a - \sigma'_r)/2)$) was studied. Where σ'_a is axial stress and σ'_r is radial stress. Here particular focus was made weather the samples show dilatant behavior or not. This truly helped on determining the strength of the soil type. For Samples that show dilitant behavior, a traditional way of estimating shear strength from triaxial test was not applied. Instead, shear strength was determined at limiting strain rate of 2%, 4% and peak pore pressure. The graphic way of illustration can be shown in Fig-16.

For long stability analysis, it is worth to have effective reliable strength parameters (ϕ' and a). It was determined from triaxial test based on the relation mean stress $(\sigma'_a + \sigma'_r)/2$ vs shear stress $((\sigma'_a - \sigma'_r)/2)$.

Friction angle was also determined based from CPTu data for comparison with the use of bearing capacity number ($N_m = q_{net}/\sigma'_{vo}$) and pore pressure parameter B_q. The degree of consolidation was taken into consideration for the proper selection of chart which is based on angle of plastifcation, β.

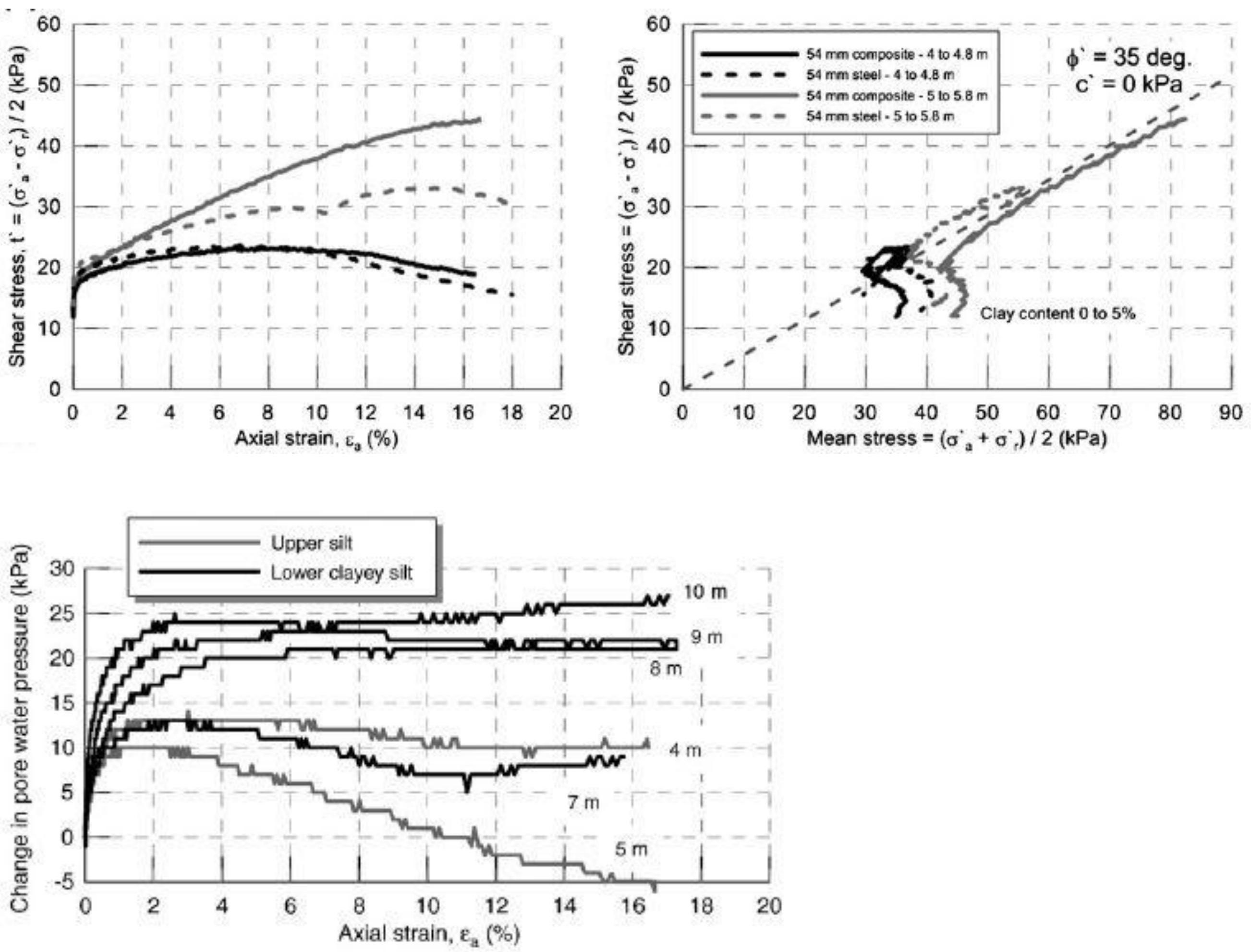


Figure 16 Triaxial test results with shear stress-shear strain, shear stress-mean stress for silty sediment and pore pressures outcome at different depths (Long et al., 2010).

3.4.4.2 Deformation and consolidation characteristics

It is possible to determine constrained modulus (M) of different soil types from CPTu data. General one dimensional constrained modulus for fine grained soils using, $M = 8.25 (q_t - \sigma_{vo})$ by Mayne and Kulhawy, (1990) was applied for clays as a means of comparison with constrained modulus for silty sediments. As it has already mentioned in section 2.4.5, stress-strain relationship in Fig-9 may not be applied for silty sediments due to non linearity of the result. For reliable determination value of M of silty soils, oedometer test results were interpreted based on Sandbækken et al, (1986) approach. preconsolidation stress (P'_c), constrained modulus (M), coefficient of consolidation (C_v) and permeability coefficient (k) was evaluated from constant rate of strain (CRS) oedometer tests results that depend on parametric equations given in Table-6 and their graphic way of interpretation can be seen in Fig-10.

4. RESULTS AND DATA ANALYSIS

4.1 Soil behaviour type

This section presents the results for the tests performed in CPT-pro software to identify soil types for all sites, following the test procedure presented in section 3.4.1. I_c values were computed for double check on identification of soil type. Besides, pore pressure parameter (B_q) was assessed as it is an important factor to differentiate more silty sediments (where B_q is less than 0.4) from clay soil (Senneset et al., 1982; Sandven, 2003). Results which were found from Robertson 1990 method was compared with the given theoretical I_c values and soil description from borehole data (refer Appendix-5) in the discussion part.

4.1.1 Soil behaviour type for site number CPTu-500

According to Robertson 1990 soil classification method, three zones have been identified as shown in Fig-17. The first layer is clayey silt to silty clay that range from a depth of 5.08m to 9.2m, the second layer is clay to silty clay ranging from 9.2m to 13.57m followed by sensitive fine grained soil type with a thickness of about 16.33m that ranges from 13.57m to 30m. For more reliable information on soil type I_c value were calculated using equation-7 where the first zone bounds between 2.79 and 2.95, zone 3 values range from 2.99 to 3.07. However, I_c values for sensitive fine grained soils are not available as this theory applies only from zone 2 up to zone 7. B_q was measured and ranges between 0.4 and 1.

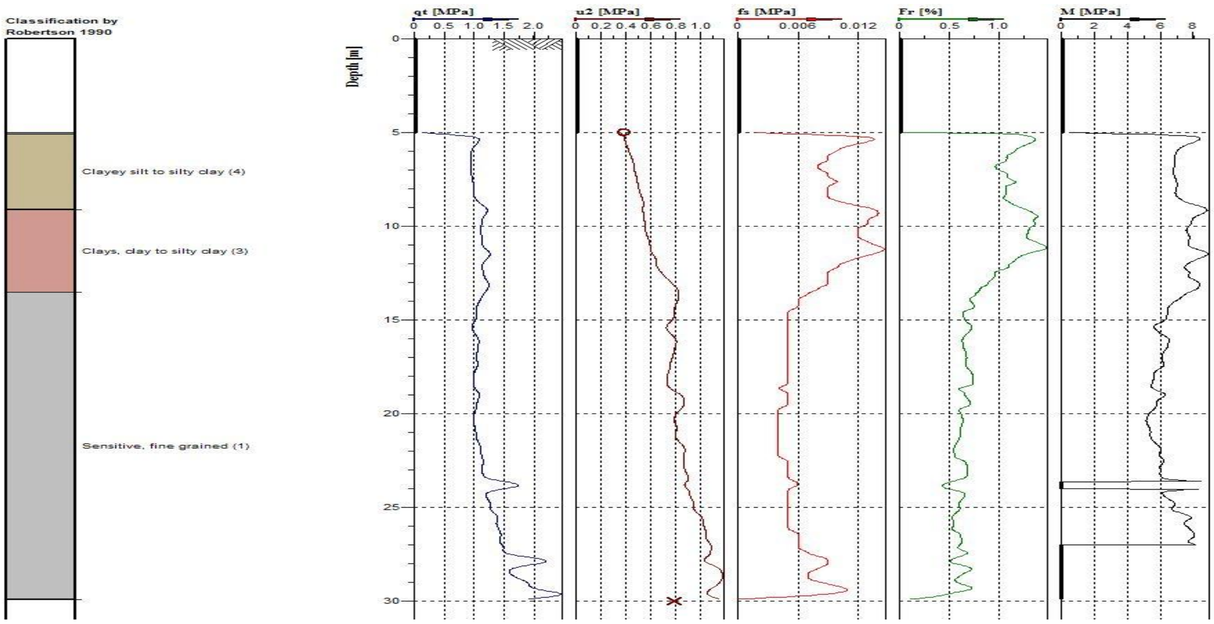


Figure 17 Log profile of CPTu-500.

4.1.2 Soil behaviour type for site number CPTu-501

This site showed the same soil stratigraphy as CPTu-500 but with a courser soil type of zone 5 at the top that ranges from a depth of 2.03m-4.03m where a thin layer of about 30cm of zone 6 is sandwiched in between (Fig-18). Depth from 9.85m to 26.81 belongs to sensitive fine grained soil type where a thin layer of clay to silty clay 60cm thick sandwiched in between. The computed value of I_c that helped for correct determination of soil type and B_q values was given in Table-8.

Table 8 Comparison of Ic values based on Qt and Fr and measurements on Bq for CPTu-501

NO	SBT, Robertson 1990	Computed I_c	Depth(m)	Theoretical I_c	Corrected soil type	B_q values
501	Silty sand-Sandy silt	2.03-2.26	2.1-4.03	2.05 – 2.6		0.06-0.07
	Clayey silt-silty clay	2.37-2.81	4.1-7.93	2.6 – 2.95	0.12m thick is part of zone-5	0.096-0.35
	Clay-Silty clay	2.82-2.93	7.97-9.80	2.95- 3.6	Clayey silt to silty clay	0.36-0.55
	Sensitive, fine grained	N/A	9.88-13.50	N/A		0.56-0.79
	Clay-Silty clay	3.21-3.28	13.60-14.13	2.95- 3.6		0.73-0.84
	Sensitive, fine grained	N/A	14.23-26.81	N/A		0.87-1.09

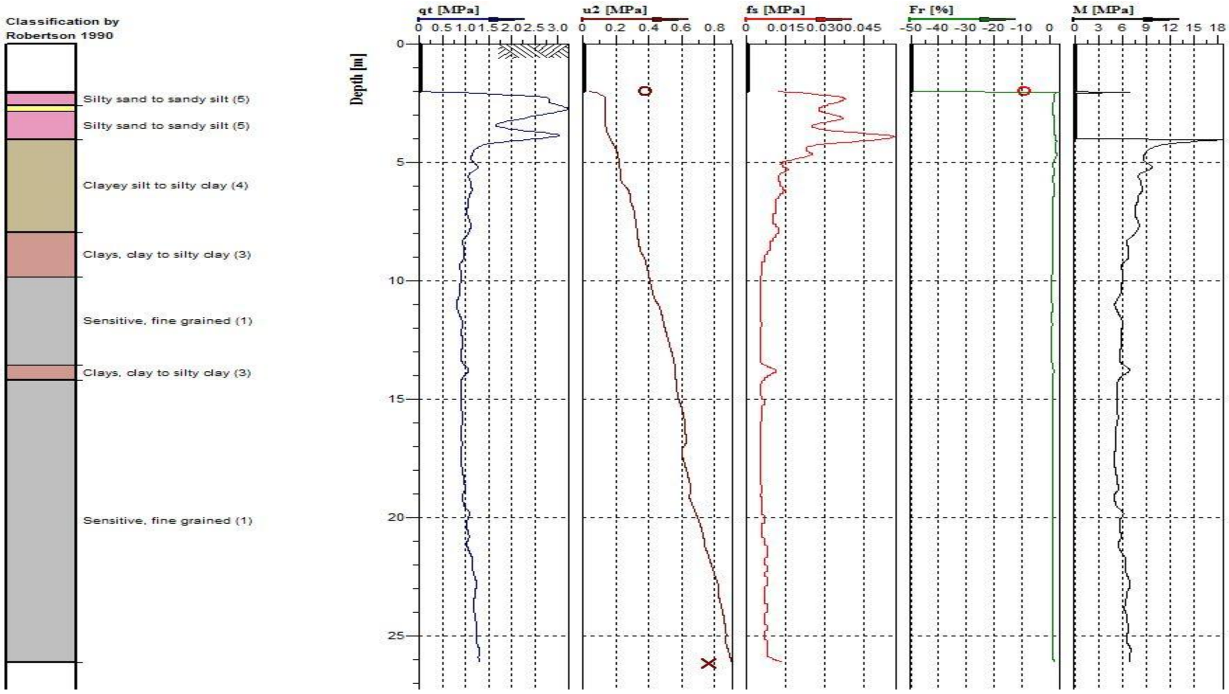


Figure 18 Log profile of CPTu-501.

4.1.3 Soil behaviour type for site number CPTu-502

This site is relatively heterogeneous and highly stratified with alternating layers of zone 1, 3, 4 and 5. But generally the dominant soil behavior type in this site (from top to bottom) based on Robertson 1990 method together with I_c values are silty sand-sandy silt, clayey silt-silt clay and clay-silty clay. The thin layers were also observed to be mingled in between the thick layers as can be shown in Fig-19. Information regarding corrected soil type and I_c values for site 502 can be referred in table-9.

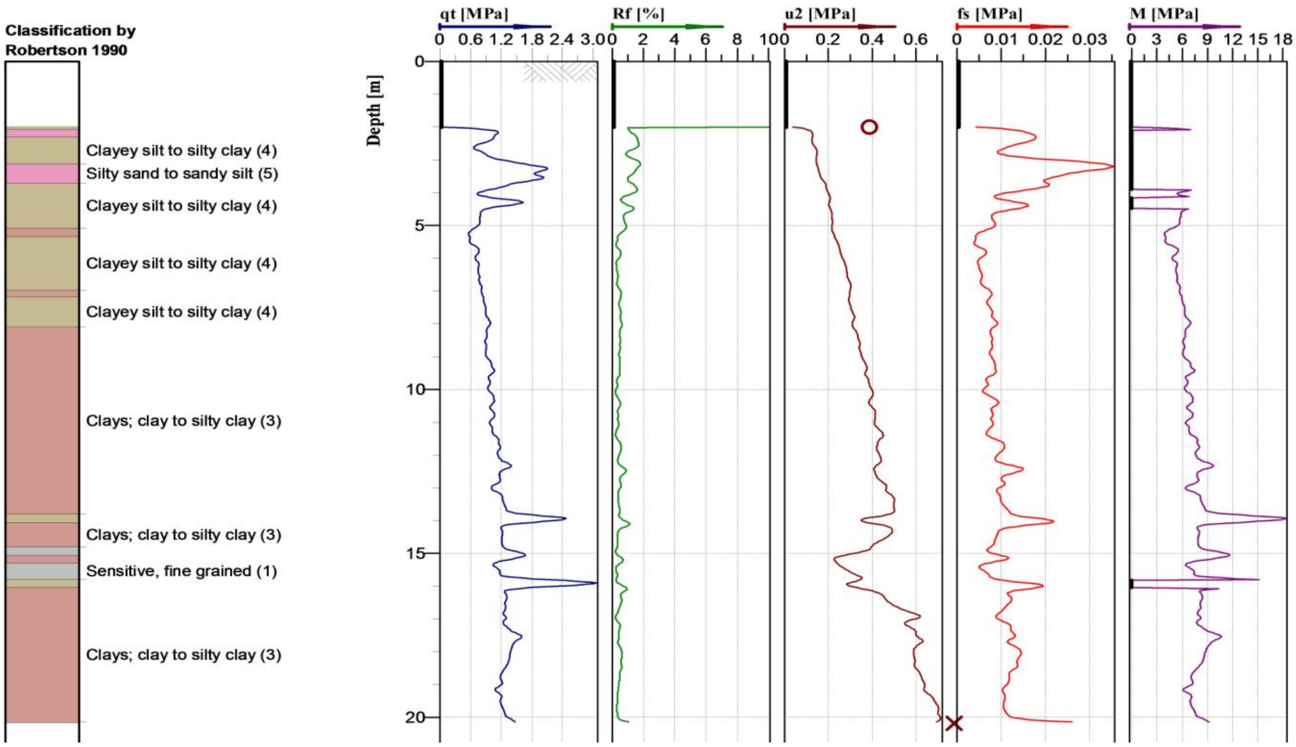


Figure 19 Log profile of CPTu-502.

Table 9 Comparison of Ic values based on Qt and Fr and measurements on Bq for CPTu-502

No	SBT, Robrtson1990	Computed I_c	Depth(m)	Theoretical I_c	Corrected soil type	B_q
1	Clayey silt to silty clay	2.89-2.44	2-2.1	2.6-2.95		0.11-0.18
2	Silty sand to sandy silt	2.44-2.52	2.1-2.3	2.05-2.6		0.11-0.12
3	Clayey silt to silty clay	2.52-2.518	2.3-3.13	2.6-2.95	Silty sand-sandy silt	0.12-0.09
4	Silty sand-sandy silt	2.518-2.56	3.13-3.73	2.95- 3.6		0.09-0.13
5	Clayey silt to silty clay	2.56-2.99	3.73-5.08	2.6-2.95		0.13-0.38
6	Clay to silty clay	2.99-2.98	5.08-5.35	2.95- 3.6		0.38-0.47
7	Clayey silt to silty clay	2.98-3.0	5.35-6.98	2.6-2.95	Clay- silty clay	0.47-0.43
8	Clay to silty clay	3.0-2.98	6.98-7.18	2.95- 3.6		0.43-0.41
9	Clayey silt to silty	2.98-2.99	7.18-8.1	2.6-2.95	Clay- silty clay	0.41-0.39

	clay					
10	Clay to silty clay	2.99-2.96	8.1-13.8	2.95- 3.6		0.39-0.35
11	Clayey silt to silty clay	2.96-2.99	13.8-14.05	2.6-2.95	Clay- silty clay	0.35-0.23
12	Clay to silty clay	2.99-3.1	14.05-14.8	2.95- 3.6		0.23-0.41
13	Sensitive fine grained	3.1-2.99	14.8-15.07	N/A		0.41-0.18
14	Clay to silty clay	2.99-3.21	15.07-15.3	2.95- 3.6		0.18-0.297
15	Sensitive fine grained	3.21-2.81	15.3-15.78	N/A		0.297-0.19
16	Clayey silt to silty clay	2.81-3.05	15.78-16.05	2.6-2.95		0.19-0.23
17	Clay to silty clay	3.05-3.34	16.05-20.13	2.95- 3.6		0.23-0.63

4.1.4 Soil behaviour type for site number CPTu-503

This site identifies four main zones such as zone 6 (clean sand to silty sand), zone4 (clayey silt to silty clay), zone3 (clay to silty clay) and zone1 (sensitive fine grained). As can be shown in Fig-20 zone 3 is associated with three thin layers of clayey silt to silty clay and one thin layer silty sand to sandy silt at depths of approximately 7.5m, 8.5m, 11m and 15m respectively. Calculated I_c values of the main soil groups are listed in table-10.

Table 10 Comparison of Ic values based on Qt and Fr and measurements on Bq for CPTu-503

NO	SBT, Robertson 1990	Computed I_c	Depth(m)	Theoretical I_c	B_q values
503	Clean sand-Silty sand	1.77-2.13	2.13-4.90	1.31 – 2.05	0.037-0.033
	Clayey silt-silty clay	2.84-2.98	5.05-6.93	2.6 – 2.95	0.15-0.26
	Clay-Silty clay	3.01-3.39	7.05-17.58	2.95- 3.6	0.26-0.34
	Sensitive, fine grained	N/A	17.73-25.17	N/A	0.34-0.44

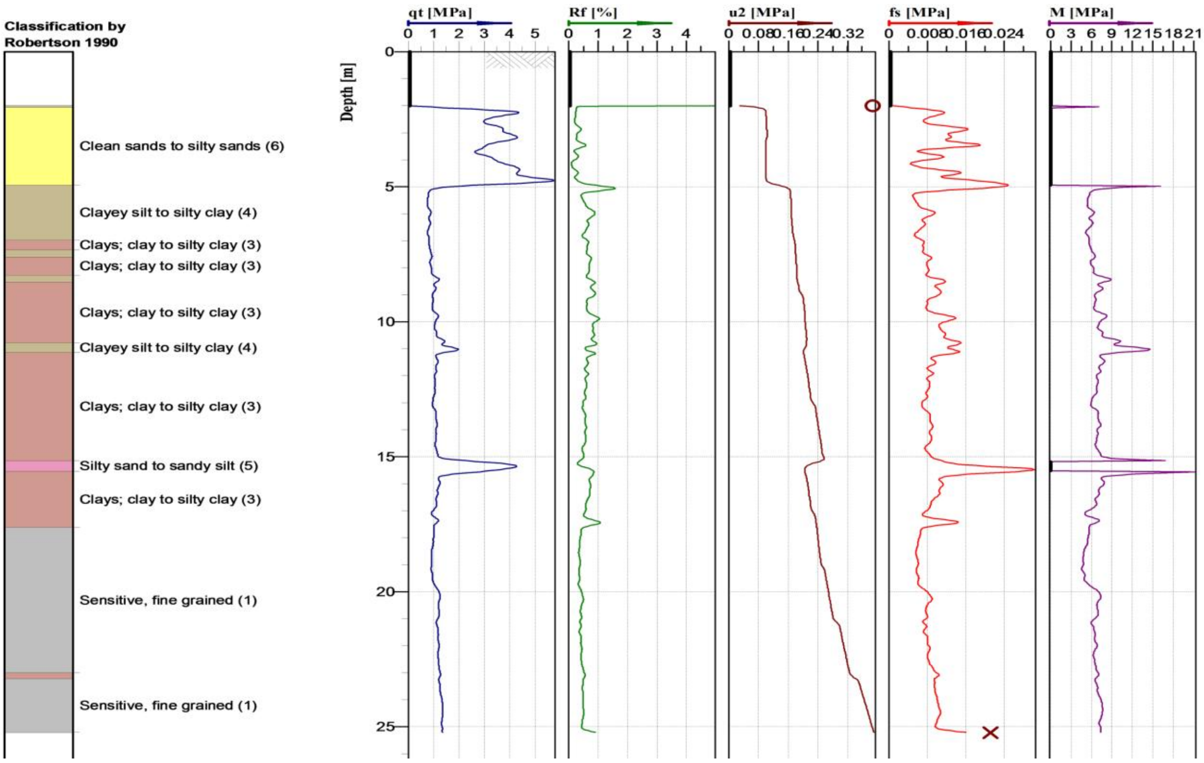


Figure 20 Log profile of CPTu-503.

4.1.5 Soil behaviour type for site number CPTu-504

Despite the variation in thickness and their occurrence at varying depth, this site showed generally the same stratigraphic sequences as of the other sites with thin layers of silty sand to sandy silt (from 3.05m-3.40m), sensitive fine grained at depth (6.50m to 6.70m and 15.30m to 15.70m) and clayey silt to silty clay at a depth of 16.40m-16.65m were identified as intercalations within the zones (refer Fig-21). I_c values of the four zones are given in Table-11.

Table 11 Comparison of Ic values based on Qt and Fr and measurements on Bq for CPTu-504

NO	SBT, Robertson 1990	Computed I_c	Depth(m)	Theoretical I_c	B_q values
504	Silty sand to Sandy silt	2.2-2.55	3.05-3.40	2.05 – 2.6	0.06-0.11
	Clayey silt-silty clay	2.8-2.96	3.50-9.13	2.6 – 2.95	0.2-0.23
	Clay-Silty clay	3.03-3.26	9.23-17.50	2.95- 3.6	0.3-0.67
	Sensitive, fine grained	N/A	17.60-25.13	N/A	0.68-0.97

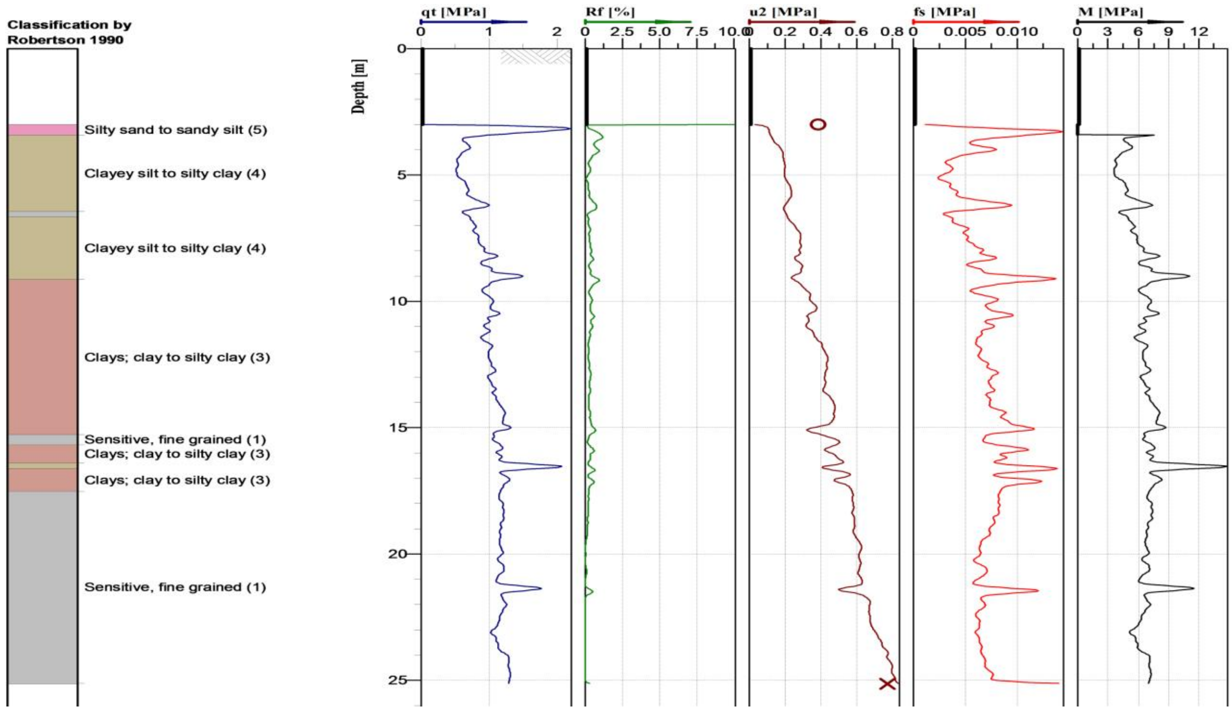


Figure 21 Log profile of CPTu-504.

4.1.6 Soil behaviour type for site number CPTu-505

This site was drilled at shallower depth as shown in Fig-22 where the first 6m thickness are of clayey silt to silty clay type (4.10m-8.02m) while the second layer is clay to silty clay (8.1m-10.18m). Their corresponding I_c value for the two layers range from 2.73-2.97 and 2.99-3.06 respectively. While B_q value range from 0.3-0.48 for clayey silt to silty clay and 0.5-0.6 for clay to silty clay layer.

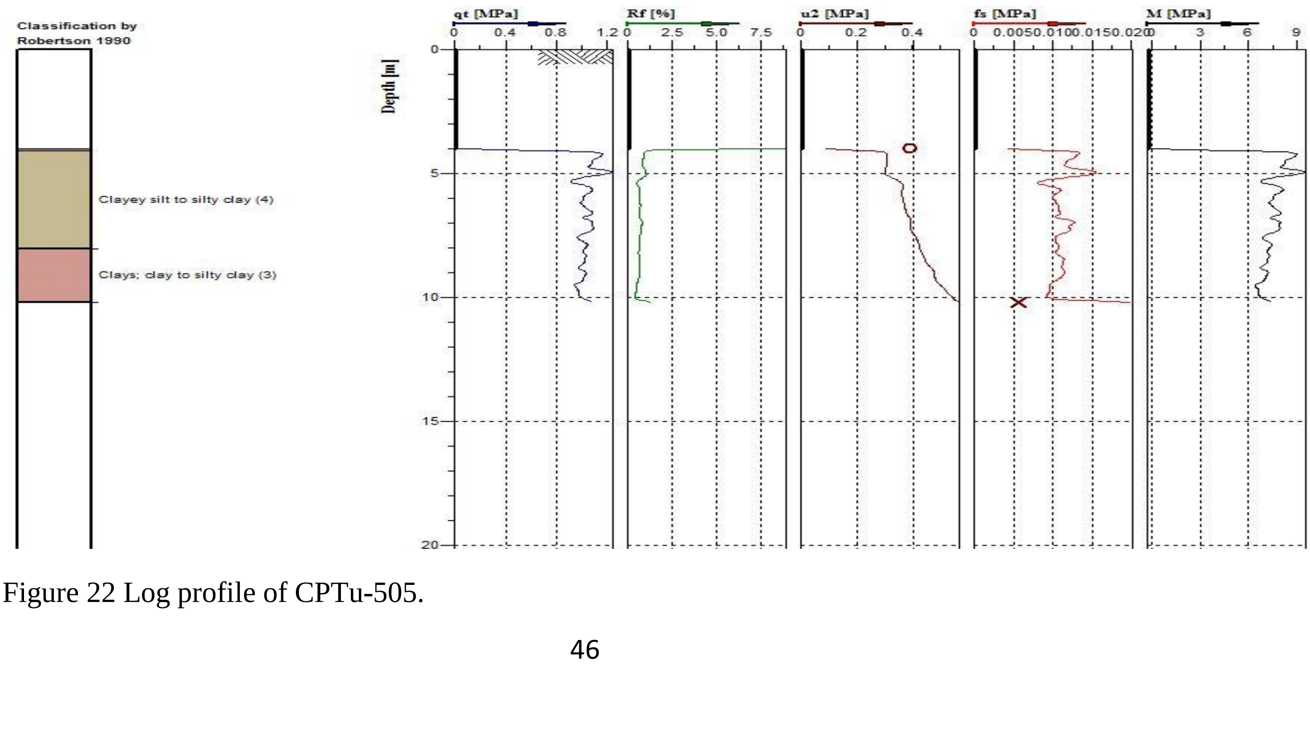


Figure 22 Log profile of CPTu-505.

4.1.7 Soil behaviour type for site number CPTu-506

Site 506 is also associated with only two soil behaviour types such as clayey silt to silty clay (5.13m-6.22m) and clay to silty clay at a depth from 6.30m to 15.60m. However, a very thin layer of organic soil peat about 0.1m is traced at the top as shown in fig-23. I_c values for the two layers are 2.99-2.97 and 3.0-3.52 respectively. Their corresponding B_q values for zone 4 ranges from 0.42-0.48 and for zone 3 from 0.49-0.86.

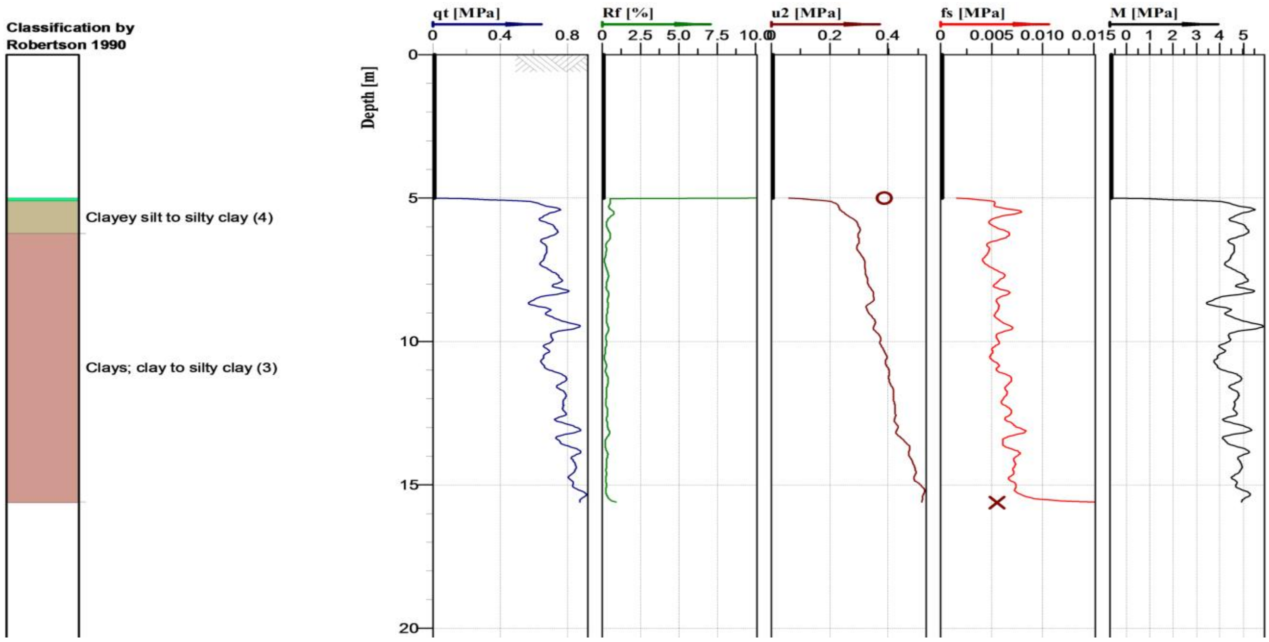


Figure 23 Log profile of CPTu-506.

4.1.8 Soil behaviour type for site number CPTu-507

Zone-4 is intercalated with thin layers of clay to silty clay at a depth of 3.0m-3.1m and 4.53m-4.80m. The general trend down depth also showed increase in clay content like the other sites as shown in Fig-24. But intercalation of clay to silty clay with in sensitive fine grained zone is also observed at depth of 12.30m-12.65m and 19.05m-20.05m. Values of I_c and B_q values for the site-507 for main layers are given in Table-12.

Table 12 Comparison of Ic values based on Qt and Fr and measurements on Bq for CPTu-507

NO	SBT, Robertson 1990	Computed I_c	Depth(m)	Theoretical I_c	B_q values
507	Clayey silt-silty clay	2.75-2.99	3.17-5.57	2.6 – 2.95	0.4-0.53
	Clay-Silty clay	3.01-3.2	5.65-11.43	2.95- 3.6	0.55-0.68
	Sensitive, fine grained	N/A	11.52-19.02	N/A	0.7-0.91
	Clay-Silty clay	3.46-3.57	19.05-20.05	2.95- 3.6	0.88-0.88

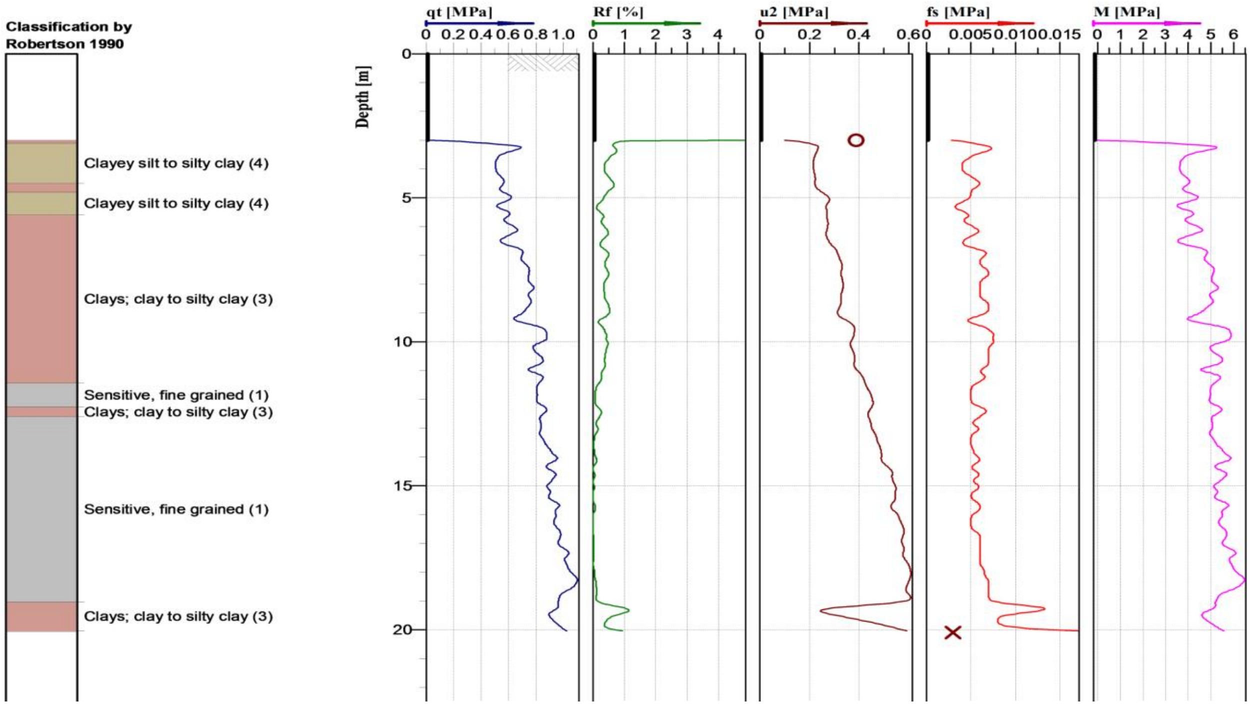


Figure 24 Log profile of CPTu-507.

4.1.9 Soil behaviour type for site number CPTu-20061298

This site is highly stratified with alternating layers of silty sand-sandy silt, clayey silt –silty clay, clay to silty clay and sensitive fine grained as can be observed both in fig-25 and table-13. And with the use of I_c values corrections to soil type was made at depths of 4.9m-5.99m and 5.99m and 6.34m.

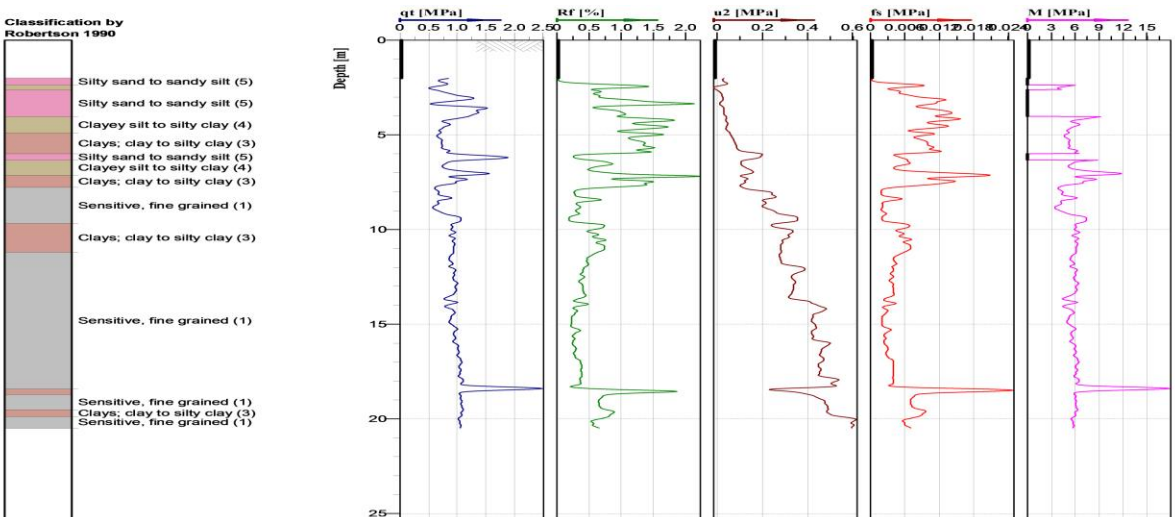


Figure 25 Log profile of CPTu-20061298.

Table 13 Comparison of Ic values based on Qt and Fr and measurements on Bq for CPTu-20061298

No	SBT, Robrtson1990	Computed I_c	Depth(m)	Theoretical I_c	Corrected soil type	B_q values
1	Silty sand to sandy silt	2.32-2.62	2-2.38	2.05 – 2.6		0.033-0.031
2	Clayey silt to silty clay	2.62-2.59	2.38-2.62	2.6 – 2.95		0.031-(-0.0054)
3	Silty sand to sandy silt	2.59-2.65	2.62-4.04	2.05 – 2.6		(-0.0054)-0.038
4	Clayey silt to silty clay	2.65-2.99	4.04-4.9	2.6 – 2.95		0.038-0.091
5	Clay to silty clay	2.99-2.69	4.9-5.99	2.95- 3.6	Clayey silt to silty clay	0.091-0.212
6	Silty sand to sandy silt	2.69-2.75	5.99-6.34	2.05 – 2.6	Clayey silt to silty clay	0.212-0.121
7	Clayey silt to silty clay	2.75-3.12	6.34-7.14	2.6 – 2.95		0.121-0.129
8	Clay to silty clay	3.12-3.13	7.14-7.78	2.95- 3.6		0.129-0.34
9	Sensitive fine grained	3.13-3.06	7.78-9.68	N/A		0.34-0.38
10	Clay to silty clay	3.06-3.1	9.68-11.2	2.95- 3.6		0.38-0.12
11	Sensitive fine grained	N/A	11.2-18.41	N/A		0.12-0.57
12	Clay to silty clay	2.9-3.37	18.41-18.72	2.95- 3.6		0.57-0.64
13	Sensitive fine grained	N/A	18.72-19.52	N/A		0.64-0.80
14	Clay to silty clay	3.38-3.42	19.52-19.88	2.95- 3.6		0.80-0.85
15	Sensitive fine grained	N/A	19.88-20.48	N/A		0.85-0.85

4.1.10 Soil behaviour type for CPTu-508

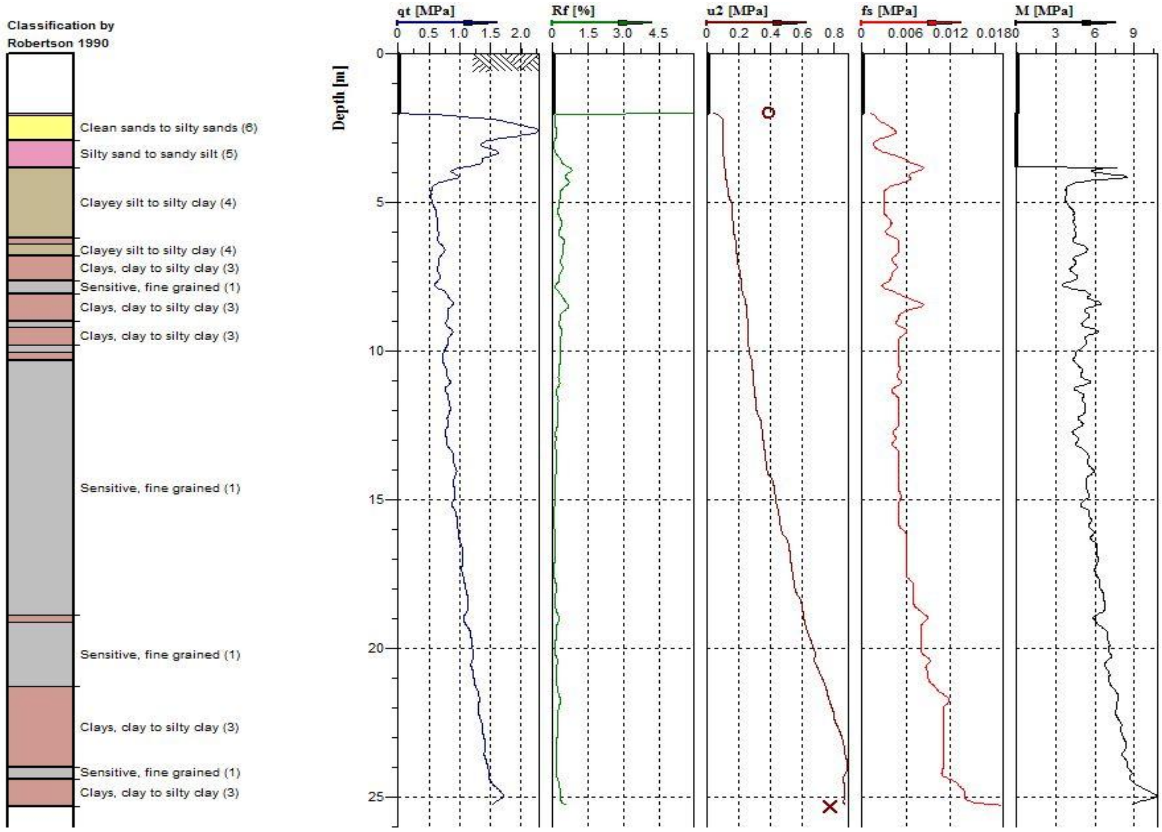


Figure 26 Log profile of CPTu-508.

This site is highly stratified with alternating layer of zone 3 and zone 1 (Fig-26). Courser soil types were found up to a depth of 3.8m with clean sand and silty sand(zone-6) at the top followed by near 0.8m silty sand and sandy silt (zone-5). B_q values were assessed like other sites and gave less than 0.4 up to a depth of around 11m. This reflects the silty nature of the strata and B_q values of 0.5 from a depth 11m-18.9m. More clayey nature of the soil is observed from a depth of 19m-25.30m with B_q values that range from 0.75-0.86.

4.2 Pore pressure measurement and unit weight

To determine the effective stress and total overburden stress, evaluation of in situ pore pressure and unit weight for each layer is important. Unit weight was computed based from soil identification while static pore pressure was estimated from piezometric data given in table-14. σ_{ov} and σ'_{ov} were calculated according to the concept mentioned in section 3.4.2.1 using equation 23, and 25 respectively.

Stress condition for some of the sites where pore pressure measurements available can be shown in Fig- 27. The in situ pore pressure for site 502 and 504 are relatively higher than the other three sites. In situ pore pressure for site 502(2-20.18m) and 504 (3.03-25.15m) varies from 15.3 to 159kPa and 7.7 to 155kPa respectively. Whereas site 506(5-15.6m), 507(3.03-20.1m) and 508(2.03-25.3m) range from 23.8 to31.4kPa and 16.9 to 48.7kPa and 11.5-34.6kPa respectively.

Table 14 Pore pressure measurements at five sites and their result on static pore pressure and stress conditions in Gunnestad-Sande

Number	**Height (m)**	**Pore pressure level(m)**	**Reading Depth (m) 27/6-11**	**Reading Depth (m) 24/9-11**	**Water height (m)**	**Static pore pressure (kPa)**	**Effective stress(kPa)**	**Total stress (kPa)**
500 North 6m	19.6	13.6	2.08	2.66	3.63	38.4		
500 South 11m	19.6	8.6	5.55	4.60	6.06	62.7		
502 North 8m	17.8	9.8	1.74	2.9	5,68	61.34	81.9	143.2
502 South 13m	17.8	4.8	2.61	3.13	10.13	101.82	130.1	231.9
506 North 11m	11.5	5.5		3.09	2.91	28.5	78.9	107.4
506 South 6m	11.5	0.5		7.94	3.06	29.98	166.1	196.1
507 North 10m	8.2	-1.8	6.45	7.03	3.55	34.8	144	178.8
507 South 5m	8.2	3.2	2.15	2.55	2.85	27.9	61.8	89.7
C2 9m	14.16	5.16		6.92	2.08	28.3	132.9	161.2
C2 15m	14.16	-0.84		7.30	7.7	75.5	192.3	267.8

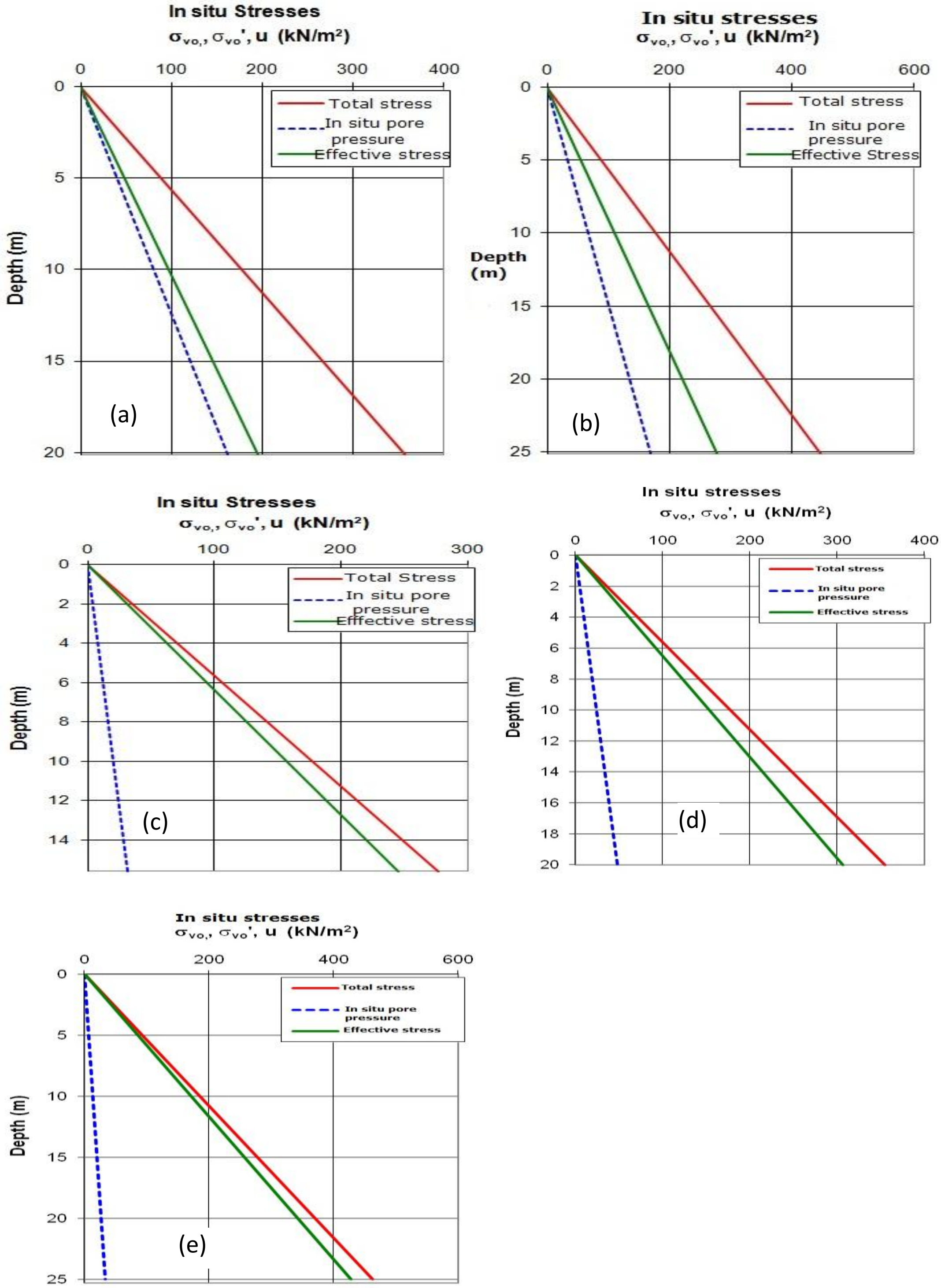


Figure 27 In situ stresses for Sites 502 (a), 504 (b), 506 (c), 507 (d), and 508 (e)

4.3 Plasticity index, Sensitivity and water content

Index test as described in table-15 illustrates that the water content of most sites varies between 30% to 37% and plasticity index from 3% to 12.5%. Low to medium sensitivity can be observed from table15 except higher sensitivity (from 23-52) was noticed for 501 at depth

9.3m and 9.6m respectively. Higher water content as compared to liquid limit is their peculiar feature of sensitive soil types which was observed in this study (Refer appendix-4). Those data helped to evaluate the general trend of plasticity index and sensitivity. According to Karlsrud et al. (2005) determination of OCR is dependent on degree of sensitivity where most of the site was analysed (for $S_t < 15$) using equations 26, 27 and 28. Those data were also crucial to define the dependency of cone factors on plasticity index and sensitivity based on equation 32 and 33.

Table 15 Index properties and their sensitivity measurements for 6-CPTu soundings

NO	Soil description	Depth(m)	Water content (%)	Plasticity index (%)	sensitivity
501	**Silt**, Clay	5.5	31	5	2
	Silty clay, sensitive	9.3	36	5	23
	Silty clay, sensitive	9.6	36	5	52
	Silty clay, sensitive	13.4	30	3	10
502	**Clay,** moderately firm With some silty layers	5.2	36.9		22
	Clay, moderately firm With some silty layers	5.6	32.9		8
	Clay, silty, moderately firm, With few sand layers	9.3	32.6	10.5	14
		9.7	34		13
	Clay, firm, silty, with few sand layers	13.4	33.3	12.5	5.3
504	**Silt,** with few sand layer	6.3			6
	Silt, with few sand layer	6.7	33	5	5
	Silt, with some organic	12.3			4
	Silt, with some organic	12.6	31	6	5
506	**Silt,**organic, with many sandy layer		32		2
	Silt,organic, with many sandy layer	5.9	35	5	2
	Silt,organic, with some sandy layer	13.7	29		2

	Silt,organic, with some sandy layer	13.9	30	3	3
507	**Silt** clay, with a sand layer	4.2	37		3
	Silt clay, with a sand layer	4.6	33	3	4
	Silt clay, with some sand layer	11.3	31		6
	Silt clay, with some sand layer	11.6	33	3	6
	Silt	13.3	31		6
	Silt	13.6	31	3	7
508	**Silt** clay	5.3	32	11	5
	Silt clay	5.7	35		5
	Silt clay	14.3	30		9
	Silt clay	14.6	30	4	10

4.4 Over consolidation ratio

OCR as an important parameter for determining cone factors and undrained shear strength was evaluated as a function of B_q, Δu, Q_t and OCR based on previous ground level. This was performed in macro-excel software for CPTu interpretation using Karlsrud et al. (2005) mentioned in 3.4.2.3. To compute OCR based on the former elevation, previous contour level for the 5 sites was determined on the basis of digital elevation model (DEM) as can be shown in Fig-30. An aging factor of 1.20 was applied for all tested sites. Estimated former elevations for site 502, 504, 506, 507 and 508 were 27m, 27m, 20m, 23m and 23m respectively. Macros excel software helped to calculate OCR based on former elevation with the following formula:

OCR= ((Previous elevation-Current elevation)*(γ)+ σ_{vo} -((Previous elevation-Current elevation +Depth)*10))/ σ'_{vo} *aging.fact Eq-38

As it is clearly seen in figure-28 OCR based on former elevation coincides with OCR based on Q_t for all tested sites except for site 502. Generally OCR based on former elevation and OCR based on Q_t gave a good match with an approximate range from 2.8 at depth 5m to 1.0 at depth of 25m with some fluctuation among them. In all sites OCR based on B_q gave the highest values with a wide range of OCR while OCR based on Δu gave the lowest values and showed narrow range of OCR with few fluctuations.

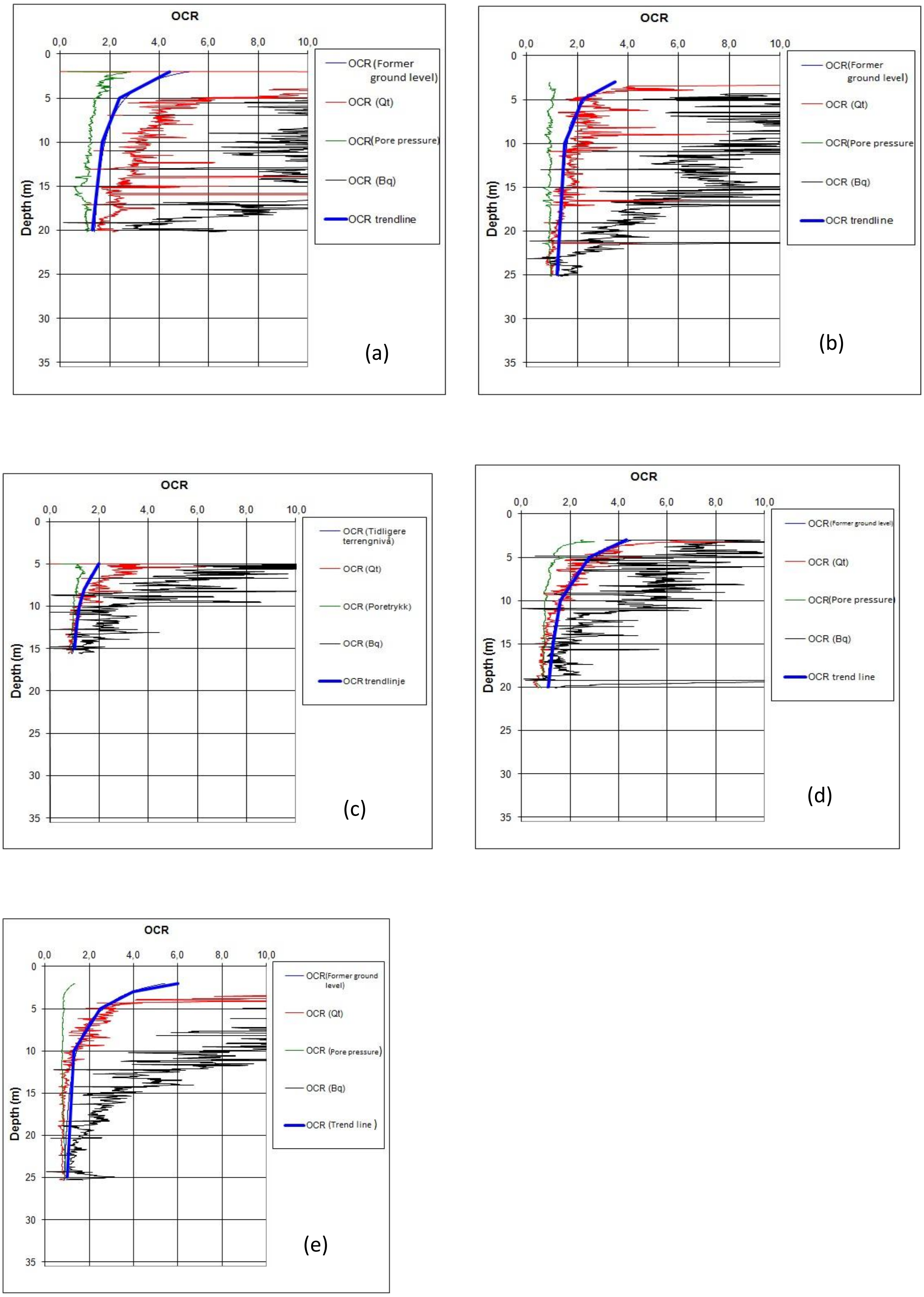


Figure 28 comparisons of OCR for site 502 (a), 504 (b), 506 (c), 507 (d), 508 (e)

For more accuracy, preconsolidation stress (p_c') at site 502 was estimated with the use of Casagranda's method as described in section 3.4.2.4. p_c' was found to be 500kPa and 700KPa at depths of 9.30m and 13.21m respectively. Once the preconsolidation stress is determined one can calculate overconsolidation ration at the two depths using equation-8 as OCR= σ'p/ σ'_{vo}. The measured effective stresses σ'_{vo} as can be shown in Fig-29, were 93kPa and 132.1kPa at depths of 9.30m and 13.21m respectively and those results an OCR values 5.37 and 5.29. One should bear in mind that preconsolidation stress was estimated using hand sketch but more accuracy can be found if it is analysed using electronic processing of the data on uniaxial compression testing with a controlled computer system (Dawidowski and Koolen, 1994).

a) b)

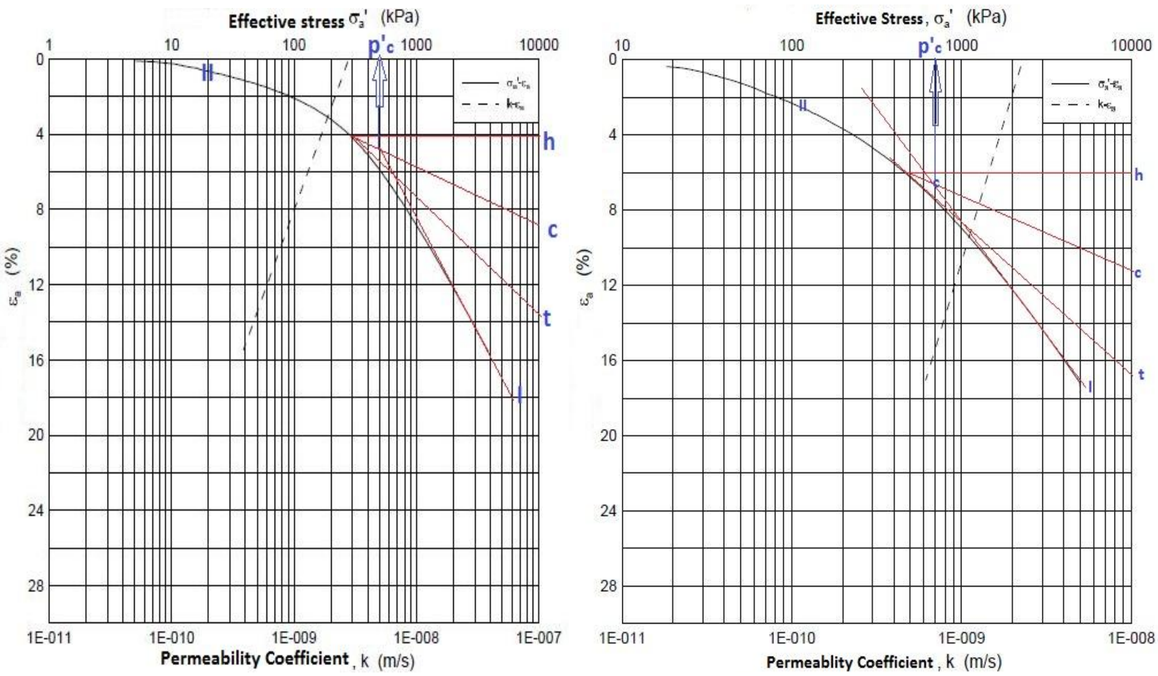


Figure 29 Estimation of preconsolidation stress (pc') using Casagrande's method for site 502 at a depth of a)9.3m and b)13.21m

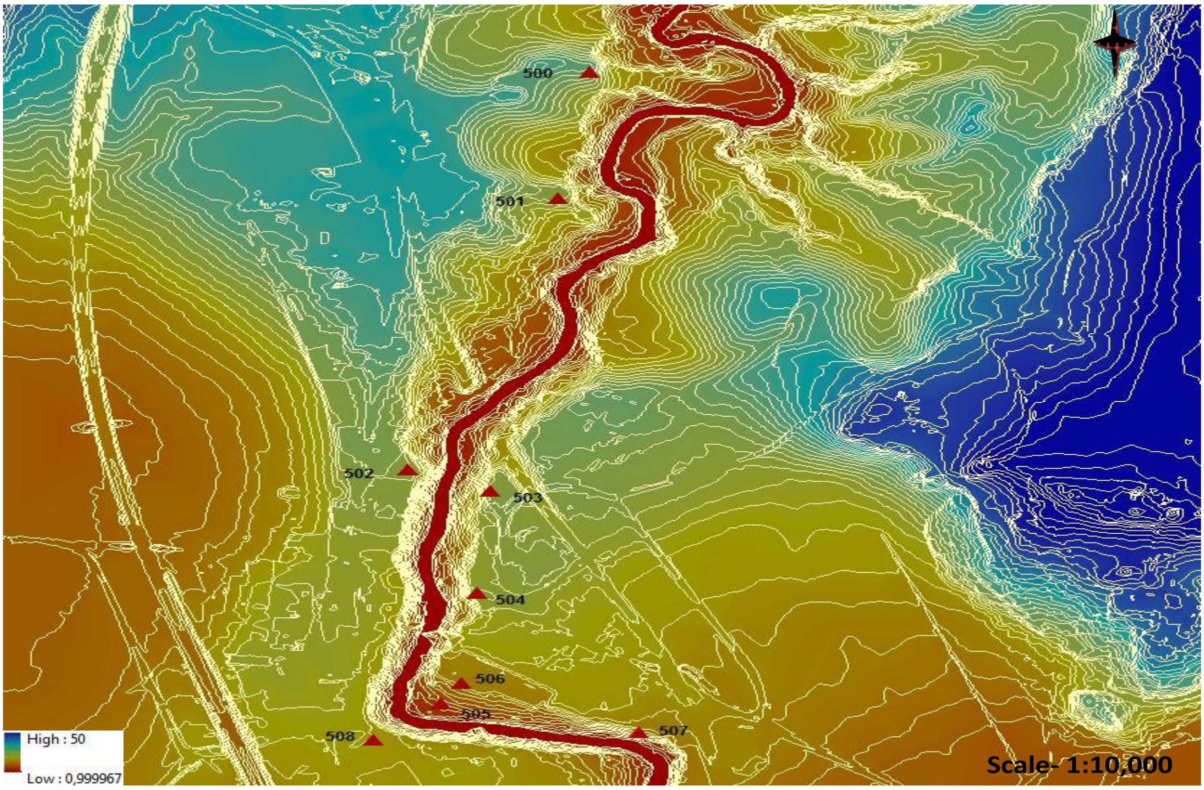


Figure 30 Digital elevation model of the study area

4.5 Cone factors (N_{kt} and $N_{\Delta u}$)

Results have been estimated for 5 sites shown in fig-31 based on Karlsrud et al., (2005) using equations 32 to 35 for sensitivity less than 15 and greater than 15. $N_{\Delta u}$ gives reliable result for clays. But for more silty sediments, cone factor evaluation based on $s_{u(CAUC)}$ for site 502 was applied (refer table-17).

Generally, N_{kt} and $N_{\Delta u}$ varies from (10.1- 8.9) and (6.9-7.2) for site 502, (9.6-8.5) and (5.1-7.0) for site 504, (9.0-8.0) and (6.0-7.1) for site 506, (9.6-8.1) and (4.6-7.0) for site 507, (10.4-8.6) and (4.4-7.6) for site 508. In order to identify the undrained shear strength of clay and silty clay sediments, N_{kt} and $N_{\Delta u}$ should be estimated. Site 502 and 504 helped to determine the undrained shear strength of silty clay and one can evaluate if N_{kt} or $N_{\Delta u}$ is preferred which is explained in discussion part section-5.3. Site 506 and 507 plays a major role in identifying cone factors and undrained shear strength of clay as the sites are homogenous clay sediments where their B_q values are greater than 0.5 as can be seen in Table-17.

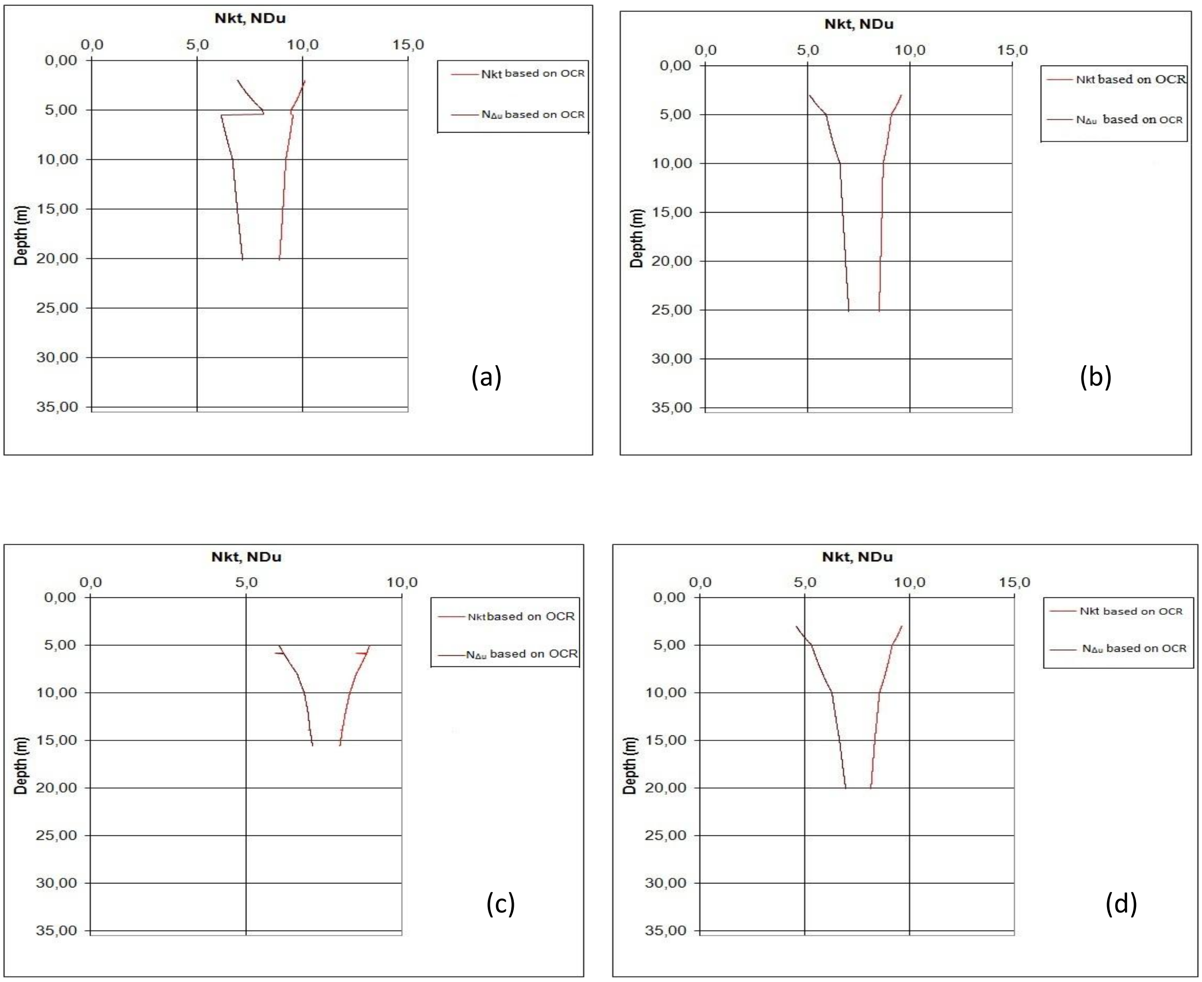


Figure 31 Results of cone factors for site 502 (a), 504 (b), 506 (c) and 507 (d)

4.6 Undrained Shear strength(s_u)

4.6.1 S_u for site 502

This site is silty clay and showed a wide range of undrained shear strength between SHANSEP, $N_{\Delta u}$ and N_{kt} based s_u. To evaluate reliable s_u for silty clay laboratory CAUC based undrained shear strength was performed at three specific depths of 5.60m, 9.43m and 13.3m with an axial stress(σ'_{ac})at 99.5kPa , 140 kPa , 199 kPa and radial stress (σ'_{rc}) at 59.8kPa, 105 kPa, 150 kPa respectively (Appendix-6). s_u was analysed at a strain rate of 2%, 4% , 10% and s_u at peak. At a depth of 5.6m s_u at 2%, 10% and s_u at peak was found 41kPa, 65kPa and 72kPa respectively.

2% strain rate was selected at a depth of 5.3m because peak pore pressure was observed at this rate. However, at a depth of 9.43m and 11.3m peak pore pressure was found at strain rate of 4% instead of 2% and their s_u became 71kPa and 98kPa respectively. s_u at peak and at 10%

strain rate was analysed for those two specific depths (9.43m and 13.3m) and gave the same s_u values of 82kPa and 120kPa respectively(refer appendix-6 and Fig-32). $N_{\Delta u}$ and SHANSEP based s_u showed similar result to each other and when it is compared with N_{kt} and CAUC based s_u considerable difference can be observed. The difference in undrained shear strength between those two regions is highly pronounced down depth. One can expect the s_u from SHANSEP model to have as good value as the one determined from the laboratory provided all factors and parameters are estimated accurately. It was found that OCR based on DEM gave lower values and the model gave lower undrained shear strength. For accurate determination on s_u, Casagrande's method was applied to estimate OCR which is one of the main parameter in the SHANSEP model (refer section 4.4). As a result the computed s_u at a depth of 9.30m and 13.21m gave 71kPa and 100.5kPa respectively. When those values compared with s_u from CAUC triaxial test at depth 9.43m and 13.30m, the same result was found. For further information of the site, it is explained in discussion part section-5.4.

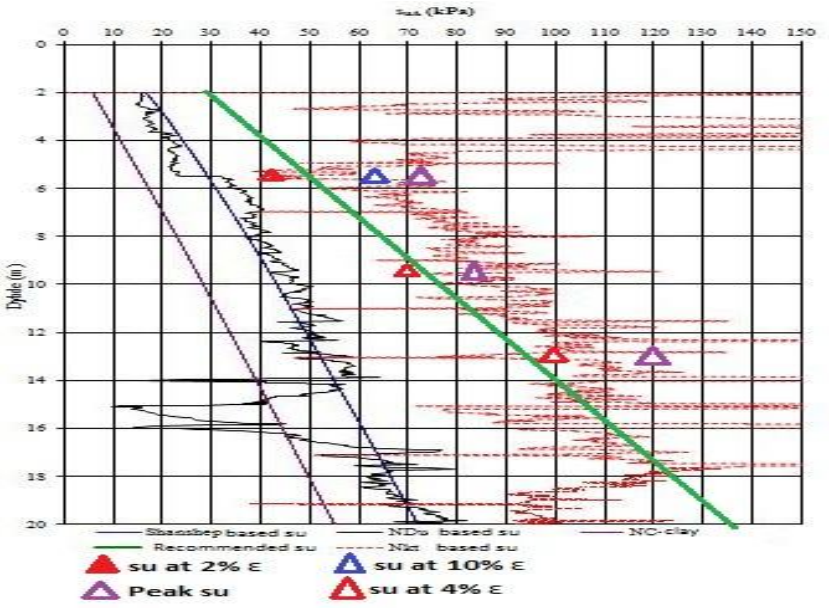


Figure 32 Estimation of su for site 502 based on su from CPTu data, su(CAUC) and SHANSEP model

4.6.2 S_u for site 504

One can refer the soil behaviour type in table-11 that silty sand to sandy silt (zone-5) found from a depth of 3m – 3.40m followed by clayey silt to silty clay (zone-4) from 3.5m – 9.13m. Their corresponding B_q values are 0.06-0.11 and 0.2-0.23 respectively. This clearly shows the silty nature of soil where determination of s_u based on $N_{\Delta u}$ and SHANSEP model may not be reliable as of undrained clayey soils. Based on $s_{u(CAUC)}$ laboratory result for site 502, it clearly indicates that N_{kt} based s_u are relatively reliable. As can be seen in Fig-33, the recommended s_u is selected at the lower limits of N_{kt} based s_u curve. But for more reliable s_u, it needs sample tests for s_u from the laboratory.

The above two layers were underlied by zone- 3 (clay –silty clay) and zone-1(sensitive, fine grained) from a depth of 9.23m-17.50m and 17.60m-25m respectively. Their corresponding B_q values were 0.3-0.67 and 0.68-0.97 respectively. So based on B_q values and nature of the soil, this section is more clayey that it would depend more on $N_{\Delta u}$ and SHANSEP based s_u. Normally one would expect SHANSEP based s_u to coincide with N_{kt} based s_u but particularly more close with $N_{\Delta u}$ based s_u. However, $N_{\Delta u}$ based s_u showed small variation with the other two methods especially at a depth less than 17m. These uncertainties might be due to:

- Pore pressure was not measured at site 504 (sea level at 13.13m), so to interpret this site pore pressure measured nearby at a site C2 was selected. The sea level of this site is 14.16m almost at the same elevation with site 504 but with 1m difference in elevation.

- Previous elevation before erosion was determined based on DEM. The estimation from the model may not give accurate estimation. For more accuracy it might need 3-D digital elevation model or other related techniques. Because of this, the recommended s_u shifts slightly to the left more close to $N_{\Delta u}$ based s_u as can be seen in Fig-33.

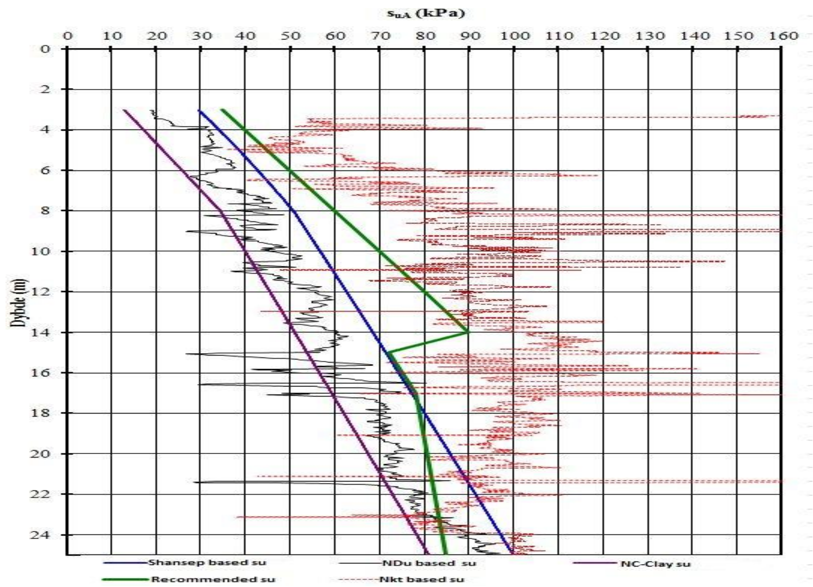


Figure 33 Estimation of su for site 504 based on su from CPTu data and SHANSEP model

4.6.3 S_u for site 506

This site can be selected as a representative model for interpreting the undrained shear strength of clay that helps to differentiate from silty clay like site 502. Homogenous clays (zone-3) were found from a depth of 6.30m-15.60m. This layer overlies by zone-4 (clayey silt to silty clay) where its depth range from 5.13m to 6.22m. Their corresponding B_q values for zone-4 and zone-3 range from 0.42-0.48 and 0.49-0.86 respectively. Soils with B_q values less than 0.4 is treated as silty soils (Lunne et .al. 1997). As a result s_u for zone-4 of this site was interpreted the same as site 504 and site 502. Here recommended s_u was selected on the bases of site 502 where values were chosen at the lower limit of N_{kt} based s_u. This specific layer also needs additional laboratory test for more accuracy (Fig-34).

For zone-3 that represents the undrained shear strength of clay show an encouraging result where SHANSEP, N_{kt} and $N_{\Delta u}$ based s_u coincides pretty well. Particularly SHANSEP and $N_{\Delta u}$ based s_u gave almost perfect match with slight low values in $N_{\Delta u}$ based s_u. This normally agrees to the theory mentioned in section-2.4.2.

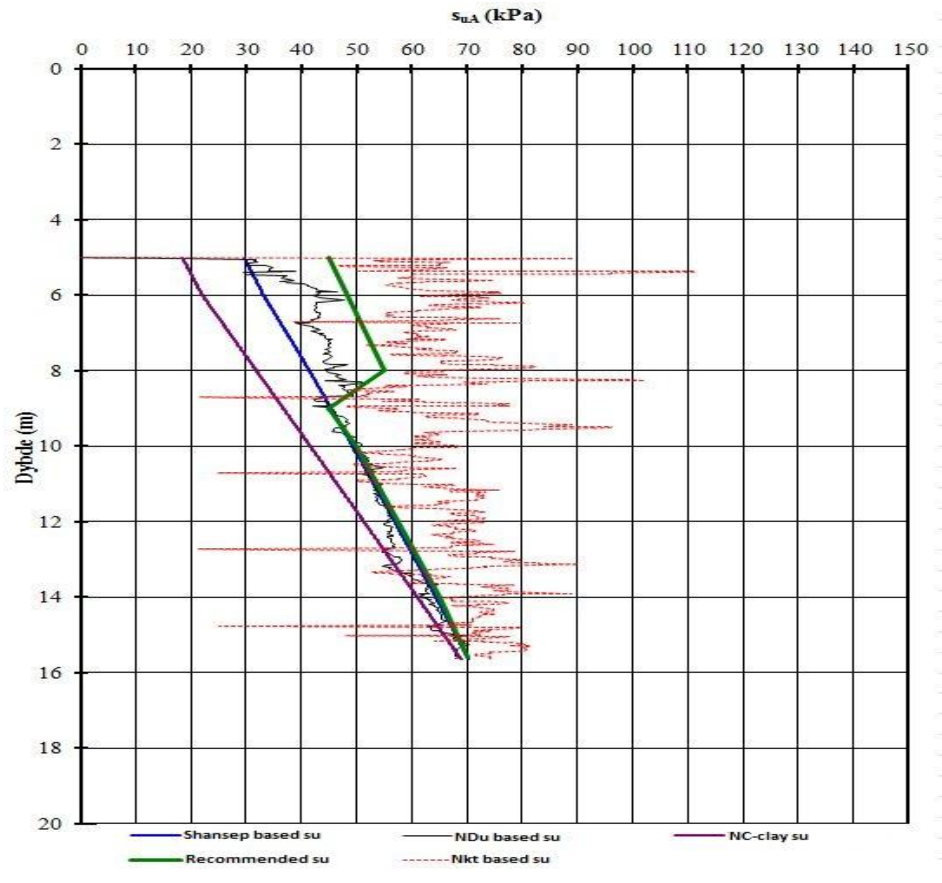


Figure 34 Estimation of su for site 506 based on su from CPTu data and SHANSEP model.

4.6.4 S_u for site 507

Site 507 is also another site where s_u clay can be determined. Zone-3 (Clay-Silty clay) was found at a depth 5.65m-11.43m and from 19.05m-20.05m. Their clayey nature of these sections can be expressed with their B_q values which is greater than 0.5 (refer Table-12). One would expect similar s_u from $N_{\Delta u}$, SHANSEP and N_{kt} but Fig-35 illustrates that SHANSEP gave the lowest value up to a depth of 8m and largest s_u from 16m to 20m. This might be due to slight under estimation of eroded soil masses. As a result the recommended s_u was chosen to be in between $N_{\Delta u}$ and N_{kt}.

Zone-4 (Clayey silt to Silty clay) and zone-1 (sensitive, fine grained) was found at depths 3.17m-5.57m and 11.52m-19.02m respectively. S_u for zone-1 was accomplished the same as s_u of clay as their B_q values are between 0.7 and 0.9. Besides, Fig-35 indicates that zone-4 the same as clayey soils where s_u from $N_{\Delta u}$ and N_{kt} nearly fit each other at depth from 3.17m-5.57m.

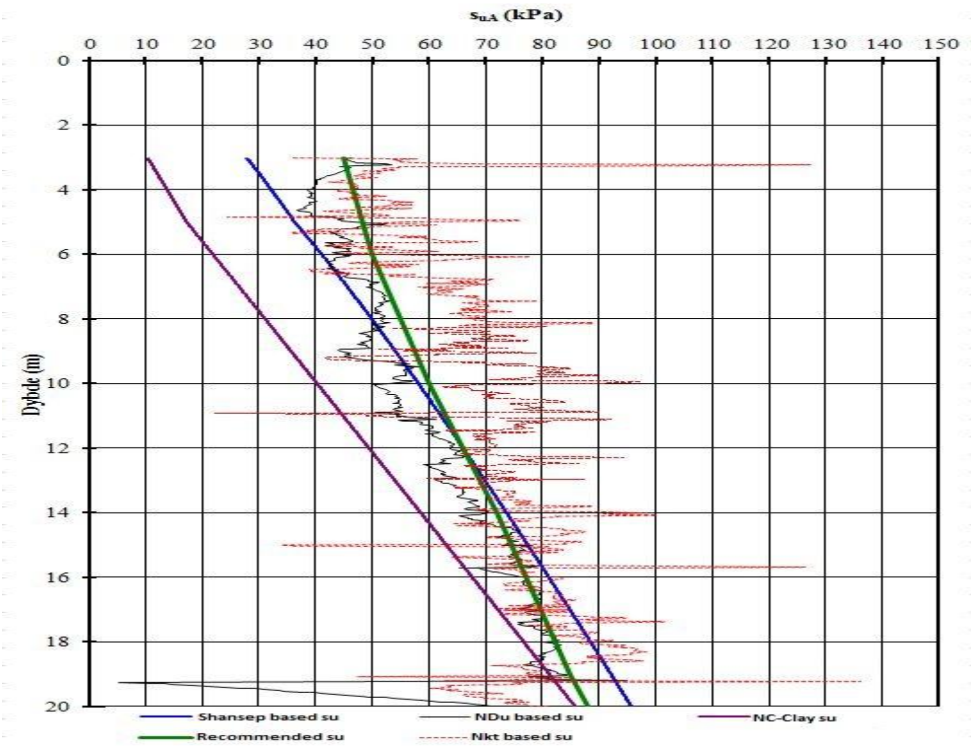
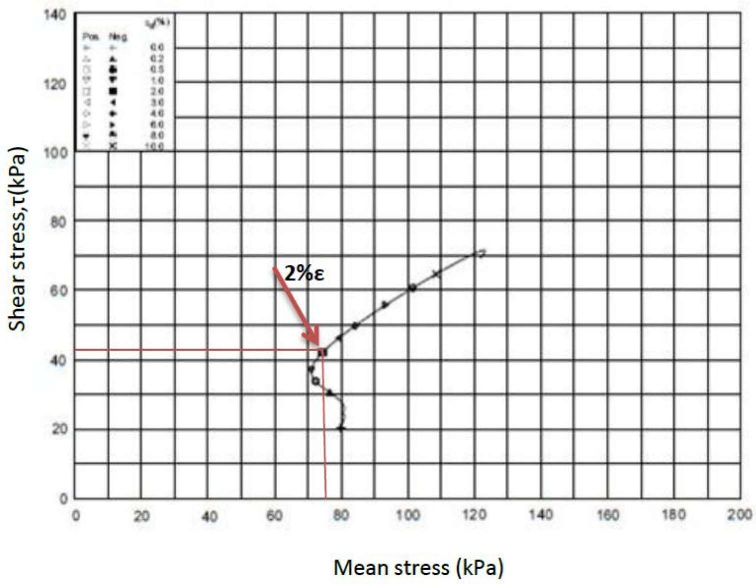


Figure 35 Estimation of su for site 507 based on su from CPTu data and SHANSEP model.

4.7 Effective friction angle (φ’) and Attraction (a)

For reliable estimation of strength parameters, CAUC triaxial test was performed for site 502 at depths of 5.6m, 9.43m and 13.3 m. the axial stress (σ_{ac}’) which was applied at the three depths were 99.5kPa, 140kPa and 199.9kPa respectively and the radial stress was 59.8kPa, 105kPa and 150kPa respectively. For the detail result done from laboratory, refer appendix-6. With those applied axial stresses, shear stress (τ) was assessed at strain (ε) rate of 0.0%, 0.2%, 0.5%, 1.0%, 2.0%, 3.0%, 4.0%, 6.0%, 8.0% and 10.0%. Fig-36(a), illustrates the maximum τ in relation to mean stresses with a strain rate of 2% at depth of 5.6m and it is 41kPa. tanϕ’ was estimated based on Mohr-Coulomb criterion by Senneset et al. (1989) in equation-19 as described in Section-2.4.4. Attraction (a) was selected as 0 for all depth mentioned. As a result tanϕ’ and ϕ’ for this depth was found to be 0.55 and 28.7°. This procedure was made for the other two depths at strain rate of 4% as can be seen in Fig-36(b) and (c). The strength parameter values are illustrated in Table-19 in discussion part including the recommended strength parameters.

(a)



(b) (c)

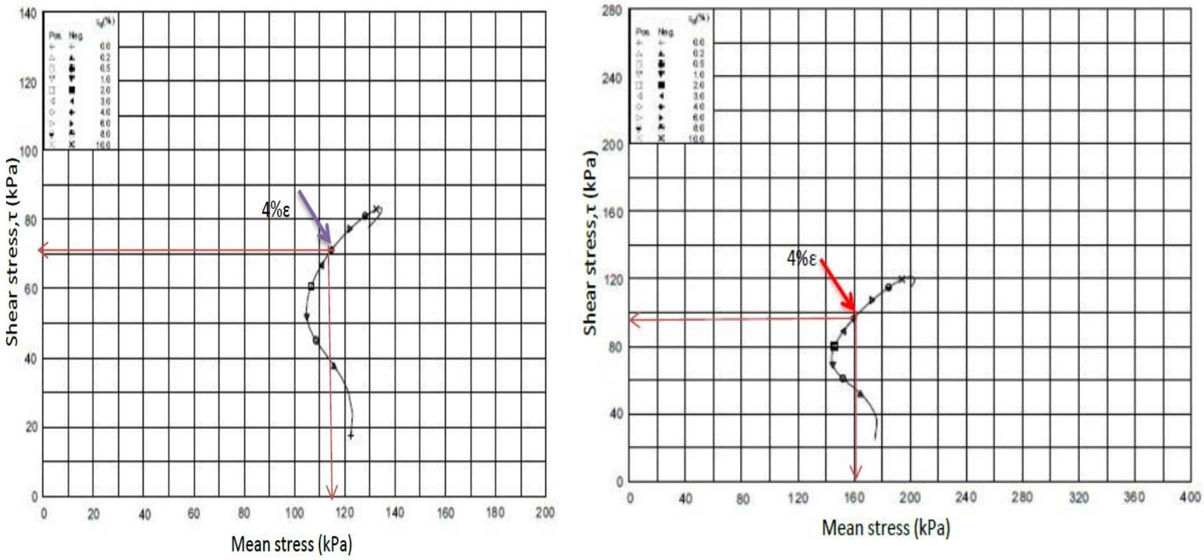


Figure 36 Triaxial test results for silty clay at depth: (a) 5.6m, (b) 9.43m and (c) 13.3m

4.8 Deformation and consolidation charachterstics

4.8.1 Deformation characteristics for silty clay

For reliable estimate of consolidation settlement on silty clay, 1-D constrained modulus, M was determined from the samples at site 502 both at a depth of 9.30m and 13.21 m. The value of M at a depth of 9.30m with σ_{vo}’ of 93kPa and at depth of 13.21 with σ_{vo}’ of 132.1kPa was estimated based on constant rate of strain from oedometer test. Their result was found as 7MPa for both depths based on stress-strain relation shown in fig-38. The result which was found from oedometer test was also compared with value of M from CPTu data using Kulhawy and Mayne, (1990) method ($M = 8.25\ (q_t - \sigma_{vo})$) and gave similar results. Constrained modulus of 6.8MPa and 8.2MPa was estimated at depth of 9.30m and 13.21m respectively.

Besides, M was evaluated on the basis of CPTu results for site 502, 504, 506, 507 and 508 using Kulhawy and Mayne, (1990) method. As can be shown in fig-37, Site 502 and site 504 showed slightly higher values on M as compared to the other sites. Higher values of M between 10MPa and 25MPa can be observed in fig-37 up to a depth of approximately 4.5m.

Generally, from a depth of 5m downward, M varies between 3MPa and 10MPa with some peak values higher than 10MPa at specific depths and no specific value of M was observed down depth.

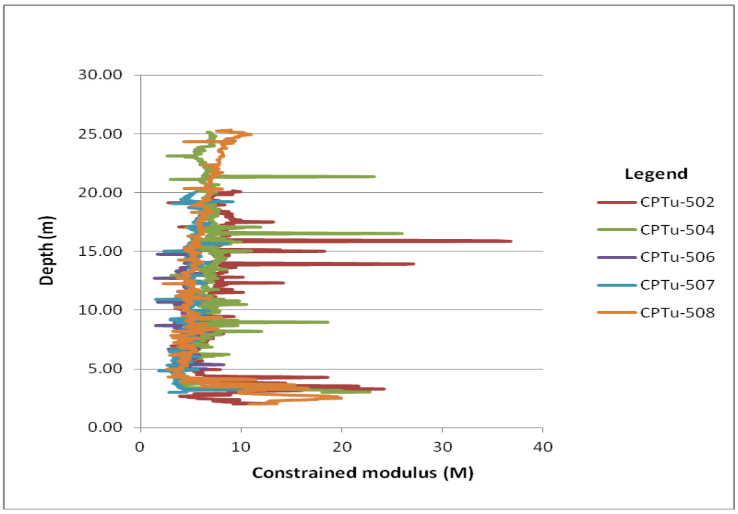


Figure 37 result on constrained modulus from CPTu data on the basis of Kulhawy and Mayne, (1990).

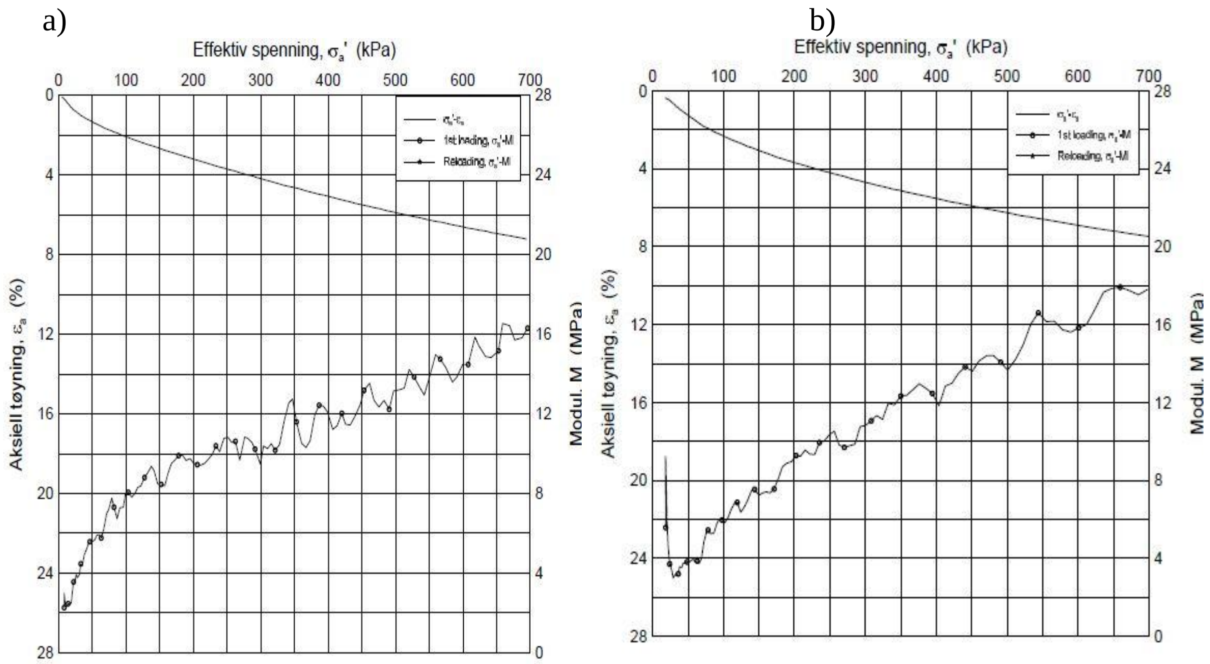


Figure 38 Result on constrained modulus from oedometer test: a) at a depth 9.30m with σvo'=93kPa and b) at a depth 13.21m with σvo'=132.1kPa

4.8.2 Consolidation charachterstics for silty clay.

In order to estimate reliable value on coefficient of consolidation, one needs to determine coefficient of permeability and constrained modulus properly. The dependency of coefficient of consolidation on both parameters was already mentioned in section 2.4.5 as $C_v = M.k/\gamma_w$. One can also determine C_v , based on dissipation data during cone penetration test However, no dissipation test was done. Coefficient of permeability was determined from oedometer test as can be shown in fig-39 and their value at a depth of 9.30m and 13.21m are $2.8x10^{-9}$m/s and $2.4x10^{-9}$m/s respectively. Those values were chosen at 0% rate of strain. Constrained modulus, M was found to be 7MPa as a result the coefficient of consolidation, c_v at depth of 9.30m and 13.21m was estimated as $2x10^{-6}m^2/s$ and $1.71x10^{-6}m^2/s$ respectively(refer fig-40).

a) b)

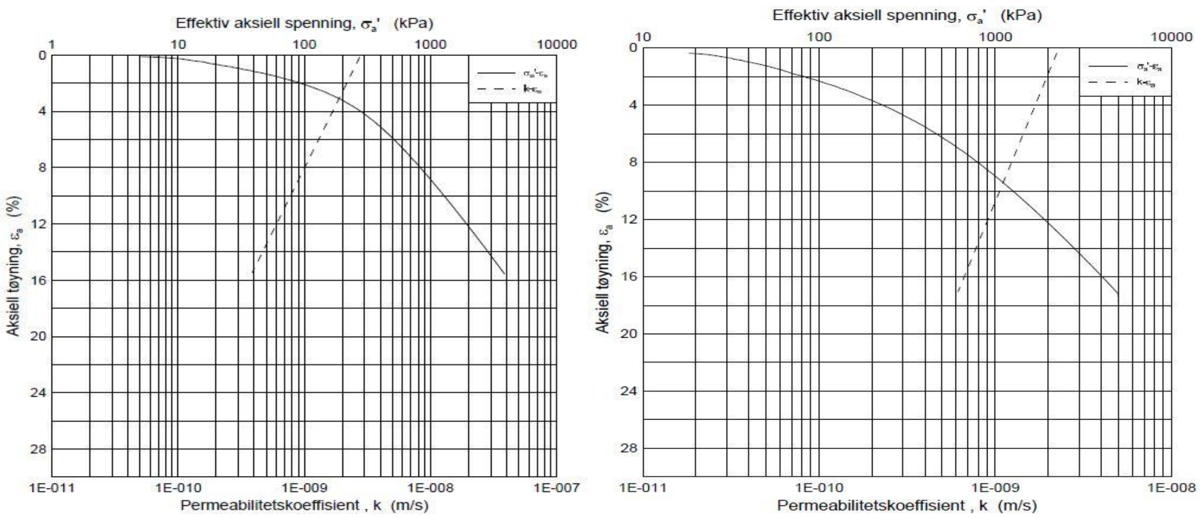


Figure 39 Result from oedometer test on coefficient of permeability, k at a depth of a) 9.30m and b) 13.21m

a) b)

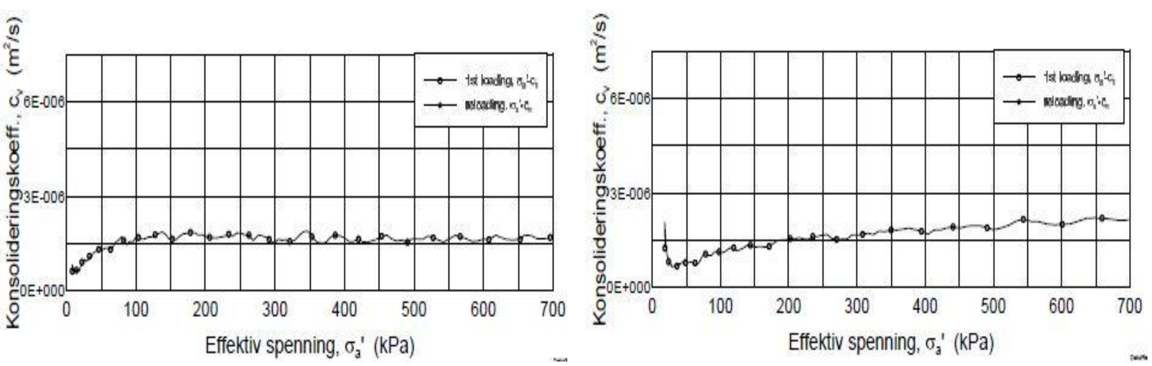


Figure 40 Result from oedometer test on coefficient of consolidation, cv at a depth of a) 9.30m and b)13.21m.

5. DISCUSSION

5.1 soil classification

Classification of Sande municipality sediments from piezocone data has been performed using the well-known SBT charts proposed by Robertson (1990), incorporating all CPTu measurements normalized with respect to vertical stress. Q_t-F_r relationship is preferred to Q_t-B_q because the latter can cause some uncertainties as complete saturation might not occur in the soils. To get a better insight into soil classification using the SBT approach, CPTu-based soil types have been compared with borehole data (appendix-5) and soil samples taken for laboratory analysis (appendix-4). Besides, computed I_c values are compared with theoretical I_c values. Borehole data and soil samples were available but not for all of the sites. However, they helped in assessing whether the soil classification methods give similar results or not.

Use of I_c values and Robertson-1990 method gave approximately the same on soil identification. Almost all the sites match the computed I_c values with the theoretical ones (table-2) except variations occur at some particular depths at sites 501, 502 and 20061298 as can be seen in tables 8, 9 and 13 respectively. More credit was given to I_c values than Robertson 1990 method as it gives 80% reliability when comparing with samples (Robertson and Cabal, 2010).

Most of augering (bore hole data) was done up to a depth of approximately 5 meter except for site 501 (drilled up to around 14 meter depth). This has been continued with soil samples tests at specific depths. Soil behaviour type for site 501 shows substantial agreements with the relevant borehole data information as cited in appendix-4 and appendix-5. Sites 504,506 and 507 also showed relatively good agreement with some variations when comparing soil behaviour type and samples taken at a specific depth.

It is worth observing that the soil behaviour method results in an alternation of silt mixtures (zone-4), clays to silty clay (zone-3), and sensitive fine grained (zone-1). In addition relatively coarse-grained soil types such as silty sand to sandy silt (zone-5) and clean sand to silty sand (zone-6) are also present at some sites on the top section of the profile. Despite the alterations, a general stratigraphic sequence from top to bottom can be mentioned as zone-4, zone-3 and zone-1 respectively and can imply general increase in clay content.

5.2 Overconsolidation ratio

Results for OCR showed that good agreement between the OCR estimated on the basis of former elevation and OCR-based on Q_t. OCR-based on former elevation is slightly lower than the one found from Q_t as can be seen in table-16. But their difference is minor and the OCR-based on former elevation can be assumed to represent the true OCR values. Fig-28(a) indicated that there is no overlap between OCRs estimated by these two methods for site 502. Attempts were made to estimate reliable values of OCR based from oedometer tests (Appendix-6) for site 502. However the stress-strain relationships at different depths do not show clear point of preconsolidation (P'_c) like plastic clays. As an alternative a commonly used Casagrande's method was applied and results an OCR of 5.37 and 5.29 at depths of 9.30m and 13.21m respectively. Q_t-based OCR at depths of 9.30m and 13.21 gave values of 3.6 and 3.0. Those values reflect that, OCR based on former elevation might be underestimated. That is, the estimation of eroded soil mass from DEM might be lower.

An interval of 5m depth is used in table-16 just to show the trend of OCR. One must bear in mind that there is no smooth trend for Q_t-based OCR and B_q based OCR. But most of the points lie between 1.0 and 3.0. Results can reflect that Q_t-based OCR has more weight as compared to Δu and B_q based OCR. OCR-based on Δu also gave very close result but slightly lower. Those findings are clearly supported on the basis of Karlsrud et al., (2005). The scattered result of OCR based on B_q was also mentioned by Mayne and Holtz, (1988); Mayne, (1991); Been, et al. (1993) and Karlsrud et al. (2005).

Normally one can expect high OCR at lower sea level as compared CPTu soundings at higher sea level being their previous sea level were the same. However this study, according the estimated previous elevation from DEM exhibits small variation in eroded material for different CPTu soundings. If one considers previous elevation (section4.4) and current elevation (table-1) for sites 504, 506, 507 and 508, relatively higher OCRs can be observed for lower sea level (sites 507 and 508) as compared sites 504 and 506. For example at a depth of 5m OCRs for sites, 504,506,507 and 508 were found to be 2.2, 2, 2.8 and 2.5 respectively. Result showed increase in OCR with decrease in sea level even though much difference was not observed between the sites.

Table 16 comparison of OCR ratio for five sites as a function of different CPTu parameters and OCR based on former elevation

	Site-502				Site-504				Site-506				Site-507				Site-508			
Depth (m)	OCR (B_q)	OCR (Δu)	OCR (Q_t)	OCR (former elevation)	OCR (B_q)	OCR (Δu)	OCR (Q_t)		OCR (B_q)	OCR (Δu)	OCR (Q_t)		OCR (B_q)	OCR (Δu)	OCR (Q_t)		OCR (B_q)	OCR (Δu)	OCR (Q_t)	
2	12	0.3	-	4.4																6.0
5	21.5	1.4	8.3	2.6	6.7	1.0	2.0	2.2	16.1	0.5	-	2.0	6.0	1.6	3.4	2.8	17.4	0.9	3.2	2.5
10	7	1.2	2.7	1.7	11.4	0.9	2.3	1.8	3.5	1.0	1.4	1.2	7.4	1.0	2.0	1.6	7.4	0.8	1.2	1.3
15	26.1	0.9	5.3	1.5	13.6	0.9	2.5	1.5	1.3	0.9	0.8	1.0	0	0.9	0.4	1.3	2.5	0.8	0.8	1.2
20	2.4	1.1	1.5	1.3	4.4	0.9	1.3	1.3					1.7	0.8	0.7	1.1	1.6	0.9	0.8	1.1
25					1.6	1.0	1.0	1.2									2.8	0.9	1	1.0

OCR (former elevation)

5.3 Cone factors (N_{kt} and $N_{\Delta u}$)

$N_{\Delta u}$ and N_{kt} for the sites were estimated on the concept of Karlsrud et al. (2005) based on the equations 32 to 35. These factors are mainly dependent on plasticity index, OCR and sensitivity. Most of them have sensitivity value less than 15, PI range from 3 to 12.5 and OCR from 1to 3. Due to small variation of those in put values, all sites listed in table-17 reflect small variation in $N_{\Delta u}$ and N_{kt} values with depth.

Sites such as 502 and 504 are associated with silty sediments which can be confirmed from B_q values. Due to their partial drainage behaviour, determination of cone factor from excess pore pressure may not be relevant. Their irrelevance was observed when comparing undrained shear strength using CAUC method with N_{kt} and $N_{\Delta u}$ based undrained shear strength. $s_{u(CAUC)}$ for site 502 at depths given in table-17. Those values are more close to N_{kt} based undrained shear strength which is an indication that N_{kt} is more reliable than $N_{\Delta u}$ in achieving reasonable s_u. For more accuracy, N_{kt} was computed based on $s_{u(CAUC)}$ and gave values listed in table-17. Reference s_u applied for site 502 at depths of 5.6m, 9.43 and 13.3 are 41kPa, 70kPa and 99kPa respectively.

Sites such as 506 and 507 are best to represent clay soils which would help to differentiate from silty sediments. According Karlsrud et al., (2005) more credit is given to $N_{\Delta u}$. The estimated values for site 506 and site 507 vary from 6.1 to 7.1 and 4.6 to 6.9 respectively. N_{kt} is computed as a means of comparison (refer table-17).

Table 17 values of cone factors at different depth for four sites including Nkt estimated from su(CAUC) for site 502

Site-502				**Site-504**				**Site-506**				**Site-507**			
Depth (m)	NΔu	Nkt	Nkt CAUC su	Depth (m)	NΔu	Nkt	B_q	Depth (m)	NΔu	Nkt	B_q	Depth (m)	NΔu	Nkt	B_q
5.6	6.1	9.5	12.2	3.05-3.4	5.1-5.2	9.6-9.5	0.06-0.11	5.13-6.22	6.1-6.3	8.9-8.8	0.42-0.48	3.17-5.57	4.6-5.4	9.6-9.1	0.4-0.53
9.43	6.6	9.2	14.3	3.5-9.13	5.3-6.4	9.5-8.8	0.2-0.23	6.3-15.6	6.3-7.1	8.8-8.0	0.49-0.86	5.65-11.43	5.4-6.4	9.1-8.5	0.55-0.68
13.3	6.8	9.1	9.85	9.23-17.5	6.4-6.8	8.8-8.6	0.3-0.67					11.52-19.02	6.4-6.9	8.5-8.2	0.7-0.91
				17.6-25.13	6.8-7.0	8.6-8.5	0.34-0.43					19.05-20.05	6.9	8.2-8.1	0.88

5.4 Undrained shear strength (s_u) for silty clay

Silty clay type of soils can be found on the border area between being classified as silt or clay. That is, small variations in grain size distribution can determine whether the material behaves drained or undrained under loading to failure.

Site 502 was the main site to understand the true nature of silty clay in the area. Pore pressure ratio (B_q) results confirmed that this type of soil could not be treated as undrained clay soils. The silty nature can be clearly observed in Fig-32 where the value of $N_{\Delta u}$ and N_{kt} based s_u show large difference. Variation in s_u among these two creates uncertainty in evaluating the reliable s_u. As a result, for more reliable estimate of s_u of silty clay, $s_{u(CAUC)}$ test was done at depths mentioned in section-4.6.1. Dilatancy effects (shown in fig-36) were also an indication not to treat them as clay soils. For silty soils, Long, et al. (2010) clearly stated that s_u can be obtained with limiting strain of about 2% or at peak pore pressure. This theory agrees with the result found at a depth of 5.6m with s_u of 41kPa. However, s_u at depths 9.43m and 13.13m, 4% strain was selected instead of at 2%. This was done because the peak pore pressure was found at 4%. Recommended s_u was selected on the bases s_u from 4% strain which was measured at depths of 9.43m and 13.3m. Based from the recommended s_u, at depth 5.6m s_u of about 50kPa was selected. The recommended one is selected considering $s_{u(CAUC)}$ test was done under undisturbed conditions. If there is some disturbance in the sample, $s_{u(CAUC)}$ could be even higher.

For comparison, estimates of s_u at peak and at 10% strain were made. These gave higher values as explained in the result section-4.6.1. When $s_{u(CAUC)}$ results compared with $N_{\Delta u}$, SHANSEP and N_{kt} based s_u , it gave similar result with N_{kt} based s_u. This correlation can be helpful in similar type of soils if laboratory test is not done. For example recommended s_u for site 504 was done based on the concept made for site 502 and the trend was selected on the lower limit of the N_{kt}-based s_u for conservative estimates. For accurate measurement of this site, $s_{u(CAUC)}$ should be performed at depths of interest. The low value of undrained shear strength from SHANSEP model was due to under estimating OCR that was obtained on the basis of digital elevation model. If OCR found from Casagrande's was applied into the model at a depth of 9.30m and 13.21m, the same result was observed.

Results of N_{kt}, $N_{\Delta u}$ and SHANSEP-based s_u which was done for clay types at sites 506 and 507. Site 506 showed small differences among them however for site 507 the value SHANSEP-based s_u showed some variations. Section-4.6.3 and section-4.6.4 explained how the recommended s_u was selected. Those two sites are good models for undrained clayey soils to clearly differentiate from silty clay where their pore pressure value (B_q) is greater than 0.5 (Long, et al. 2010). With the use of Karlsrud et al. (1997) method $N_{\Delta u}$-based s_u from CPTu data at both sites gave reliable results. This was compared with the concept of Ladd et al. (1977) SHANSEP model. Those two models allowed in interpreting the trend of recommended undrained shear strength as can be seen in Table-18.

Table 18 Recommended undrained shear strength for four sites in relation to depth

Site-502			Site-504			Site-506			Site-507		
Depth (m)	Rec*S_u (kPa)	Soil type	Depth (m)	Rec*S_u (kPa)	Soil type	Depth (m)	Rec*S_u (kPa)	Soil type	Depth (m)	Rec*S_u (kPa)	Soil type
2.1	30	Zone-5	3.1	35	Zone-5	5.5	47	Zone-4	3.5	46	Zone-4
2.5	32.5	Zone-5	3.4	37	Zone-5	6.0	49	Zone-4	5.0	48	Zone-4
3.0	35	Zone-5	4.0	40	Zone-4	8.0	55	Zone-3	6.0	50	Zone-3
4.0	40	Zone-4	9.0	65	Zone-4	10	50	Zone-3	8.0	55	Zone-3
6.0	52	Zone-3	12	80	Zone-3	12	56	Zone-3	10	60	Zone-3
8.0	65	Zone-3	15	72	Zone-3	14	65	Zone-3	12	65	Zone-1
12	88	Zone-3	17	78	Zone-3	15.6	70	Zone-3	14	72	Zone-1
15	105	Zone-1	19	80	Zone-1				16	75	Zone-1
15.5	106	Zone-1	21	82	Zone-1				18	82	Zone-1
17	116	Zone-3	23	84	Zone-1				19.5	86	Zone-3
20	135	Zone-3	25	86	Zone-1				20	88	Zone-3

Rec*s_u = Recommended S_u

5.5 Effective friction angle (φ') and Attraction (a)

It is already mentioned in chapter one that site 502 is classified in risk "zone-4" with medium hazard level and consequence class, "very serious". This would definitely demand reliable effective strength parameters for long term stability analyses with conservative estimates. If one takes a look at Fig-36, the graphs show dilatant behaviour which is the characteristic feature of silty soils. The projections of the three mean stresses $(\sigma'_a + \sigma'_r)/2$, P' given in table 19 for the three depths, at 4% strain rate meet the axis of shear stress $(\sigma'_a - \sigma'_r)/2$ at 0kPa which indicates that attraction (a) value is also zero. This was done on the basis of Long, et al. (2010) as shown in Fig-16. The effective friction angle φ' was estimated both from triaxial tests and from CPTu data as shown in table-19. The two methods gave similar result with slightly higher value of φ' about 35^o from CPTu data as compared to an average value of 31^o from triaxial tests. Friction angle values were evaluated from CPTu data by relating the bearing capacity number $((q_{net}/\ \sigma'_{vo}) = N_m)$ with pore pressure parameter, B_q. Angle of plastification was assumed to be zero by taking a chart for lightly overconsolidated silts

(Senneset et al., 1982 and Senneset et al., 1988). The values from CPTu data were made for comparison. However for reliable estimation results from triaxial tests were used. Friction angle was determined at shear stress τ, with 4%ε and gave ϕ′ values as can be shown in table-19.

Table 19 Estimation of strength parameters based on CPTu data and CAUC laboratory tests for site 502

Result from CAUC laboratory test						**Result from CPTu data**			
Depth (m)	τ (4%ε) (kPa)	P' (kPa)	a	tan ϕ′	ϕ′ (°)	B_q	N_m	tan ϕ′	ϕ′
5.60	50	85	0	0.59	30.5	0.4	8.72	0.72	35.7
9.43	71	115	0	0.62	31.8	0.3	10.43	0.71	35.4
13.30	98	160	0	0.61	31.4	0.4	7.33	0.68	34.2

5.6 Deformation and consolidation characteristics

For the reasonable prediction of deformation constrained modulus M for silty clay, that need to estimate the magnitude of consolidation settlement, CPTu data was analysed with the concept of Kulhawy and Mayne, (1990) method. In addition laboratory test was undertaken to establish reference soil parameters. Both methods gave very similar result which indicated that Kulhawy and Mayne, (1990) method can be reliable result in the absence of laboratory test. Besides, one can gather information on constrained modulus for different layers with the help of continuous measurements from CPTu which maximize time efficiency. As can be shown in Table-20 and Fig-37, there are range of M values for different soil behavior types. Based on CPTu results, the range of constrained modulus for silty clay was between 4MPa and 12MPa. This indicated that value of M cannot be represented by one specific value however the average value between the depth of 3.73m and 20.1m was found to be 7.5MPa and can be reasonable estimation to define deformation behavior of silty caly at this site.

As mentioned earlier in section 4.8.2, consolidation characteristics depend on hydraulic conductivity and constrained modulus M. Based on result of M from CPTu data and the oedometer results for k and M at a depth of 9.30m and 13.21m, both gave similar result on coefficient of consolidation ($2x10^{-6}m^2/s$ and $1.71x10^{-6}m^2/s$ respectively). The consolidation characteristics of silty clay at site 502 can have a range of value on coefficient of consolidation due to range of values in M from 4.0MPa to 12MPa and very small variability in hydraulic conductivity, k. But one should bear in mind that the difference in constrained modulus and coefficient of consolidation down depth is not much exaggerated for silty clay soil and can be represented with an average estimate of about $1.84x10^{-6}m^2/s$ taking an average value of M, 7.5MPa and with k value $2.4x10^{-9}m/s$ which was obtained from oedometer.

Table 20 Estimated range of constrained modulus based on CPTu data for SBT zones at four sites

SBT-502	**Depth(m)**	**M (MPa)**	**SBT-504**	**Depth(m)**	**M(MPa)**	**SBT-506**	**Depth(m)**	**M(MPa)**	**SBT-507**	**Depth(m)**	**M(MPa)**
Zone-4	2-2.1	9.0-11.5	Zone-5	3.05-3.40	8.0-18	Zone-4	5.13-6.22	5.0-7.0	Zone-4	3.17-5.57	2.0-5.5
Zone-5	2.13-3.73	8.0-10	Zone-4	3.50-9.13	4.0-9.7	Zone-3	6.30-15.6	4.0-7.0	Zone-3	5.65-11.4	3.0-7.0
Zone-4	3.73-5.08	4.0-7.66	Zone-3	9.23-17.5	5.6-11				Zone-1	11.5-19.0	4.5-7.0
Zone-3	5.08-14.8	4.0-8.7	Zone-1	17.6-25.1	5.0-7.0				Zone-3	19.05-20	3.0-6.0
Zone-3	16.1-20.1	6.8-12									

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The main objective of this work was to provide an approach for making reliable estimates of the mechanical properties of silty clay on the basis of CAUC triaxial and oedometer test results, together with CPTu interpretations, to be used as input in slope stability assessment. It includes determination of undrained shear strength (s_u), effective friction angle (ϕ') , attraction (a), deformation (in terms of constrained modulus, M) and consolidation (coefficient of consolidation, c_v) characteristics of silty clay. Some of the findings were as follows:

- CPTu data are very useful for identification of silty clay and other types of soils. A SBT chart proposed by Robertson (1990) was applied to all sites and this was correlated with SBT I_c values that gave similar results. Although the information from borehole profile was not complete; correlations with CPTu data gave very good agreement.
- The overconsolidation ratio OCR was determined on the basis of B_q, Q_t, Δu and previous elevation. B_q-based OCR gave very large and scatterd values. As a result it was used just for comparison. OCR based on previous elevation and Q_t showed similar results. This clearly indicates that the reliability of Q_t-based OCR as supported by elevation-based OCR. Δu-based OCR was also analysed and gave lower values, most of them below the normally consolidated zone.
- To have a reliable estimation of OCR at site 502, oedometer test was done. Unfortunately no clear point of preconsolidation (P_c') was noticed from the stress-strain relationship. The commonly-used Casagrande method was applied instead and the results gave a good match with the OCR based on Q_t. This indicated that the eroded mass with the use of DEM might have been underestimated for this site.
- Determination of cone factor for silty sediments like sites 502 and 504 was mainly based on N_{kt}, rather than $N_{\Delta u}$. The importance of N_{kt} was observed when comparing the reference undrained shear strength with N_{kt}- and $N_{\Delta u}$-based s_u. N_{kt}-based undrained shear strength gave similar result with the reference s_u from the laboratory. However,

for more accuracy N_{kt} for site 502 was back-calculated from reference undrained shear strength at depths of 5.6m, 9.43m and 13.3m. For clay sites such as site 506 and site 507, $N_{\Delta u}$ was chosen.

- Reliable estimation of s_u for silty clay was mainly dependent on site 502 based on the result of s_u from SHANSEP, N_{kt}, $N_{\Delta u}$ and CAUC. For this type of study recommended trend of s_u was dependent on s_u from CAUC test at a strain of 4%. Very good agreement was noticed when SHANSEP-based s_u was estimated based on OCR which was obtained from Casagrande's method. However SHANSEP-based s_u estimated from DEM-based OCR was underestimated. The lower limit of N_{kt} based s_u indicated similar result with the one found triaxial test but large difference was observed when compared with $N_{\Delta u}$-based s_u. On the other hand, it was observed that $N_{\Delta u}$-based s_u is more reliable than N_{kt}-based s_u for clay type soils.

- Friction angle (φ'), for silty clay was evaluated on the basis of CAUC triaxial tests and CPTu data. Both of them gave similar results with higher values of about 35° from CPTu data and average value of 31° from laboratory. However for reliable and conservative estimates more credit was given to the one estimated from CAUC triaxial tests. Attraction (a) was found to be 0 and it seems reasonable for soils having B_q value below 0.4.

- When comparing the deformation constrained modulus (M) for silty clay obtained from oedometer test and CPTu data (based on Kulhawy and Mayne, method), both gave nearly the same result for specific depths at site 502. This indicated that one can rely on CPTu data in the absence of laboratory tests. Even though, M computed from CPTu data did not show specific value, their variations with depth is not much exaggerated and can be represented with an average value of 7.5MPa.

- Due to small variations in constrained modulus (M) and coefficient of permeability (k) down depth, a consolidation characteristic for silty clay showed small variations with depth and was not possible to decide a specific value on coefficient of consolidation. However, the average value which is nearly the same with the laboratory result can represent the consolidation characteristics of silty clay at the site.

6.2 Recommendations for further works

The work done in this research identified several important issues that need further investigation. The following are some of the identified issues.

- To differentiate the methods to be used for assessment of undrained shear strength of clay and silty clay, the study was more focused on sites 502, 504, 506 and 507. Other CPTu soundings were not applied for determination of undrained shear strength due to lack of either pore pressure measurements from the field or sensitivity and plasticity index measurements, which are important parameters for the method applied by Karlsrud et al. (1997). I would recommend collecting additional data on the mentioned parameters so that undrained shear strength of the other sites can be computed. This will give broader knowledge on the properties of clay and silty clay.

- Although the results for strength parameters are satisfactory, the friction angle ϕ' determined from CPTu data is slightly higher than the values obtained form CAUC triaxial tests. Their difference can be due to sample disturbance or minor inaccuracy in the CPTu data. For more confidence on strength parameters, I would suggest additional CAUC triaxial tests to be applied using high quality block samples at site 502. Further work is needed for making more accurate estimation of strength parameters from CPTu data.

- Silty clay with the well known Kulhawy and Mayne, (1990) model showed a range of values on constrained modulus, M (4.0MPa to 12MPa). Oedometer result which were available only at depths of 9.30m and 13.21m and gave similar results with constrained moduli estimated from CPTu data. To confirm the small variation in M and coefficient of permeability (k) with depth, I would recommend additional oedometer tests to be taken at different depths. This can confirm whether silty clay exhibits many orders of magnitude on coefficient of consolidation or not.

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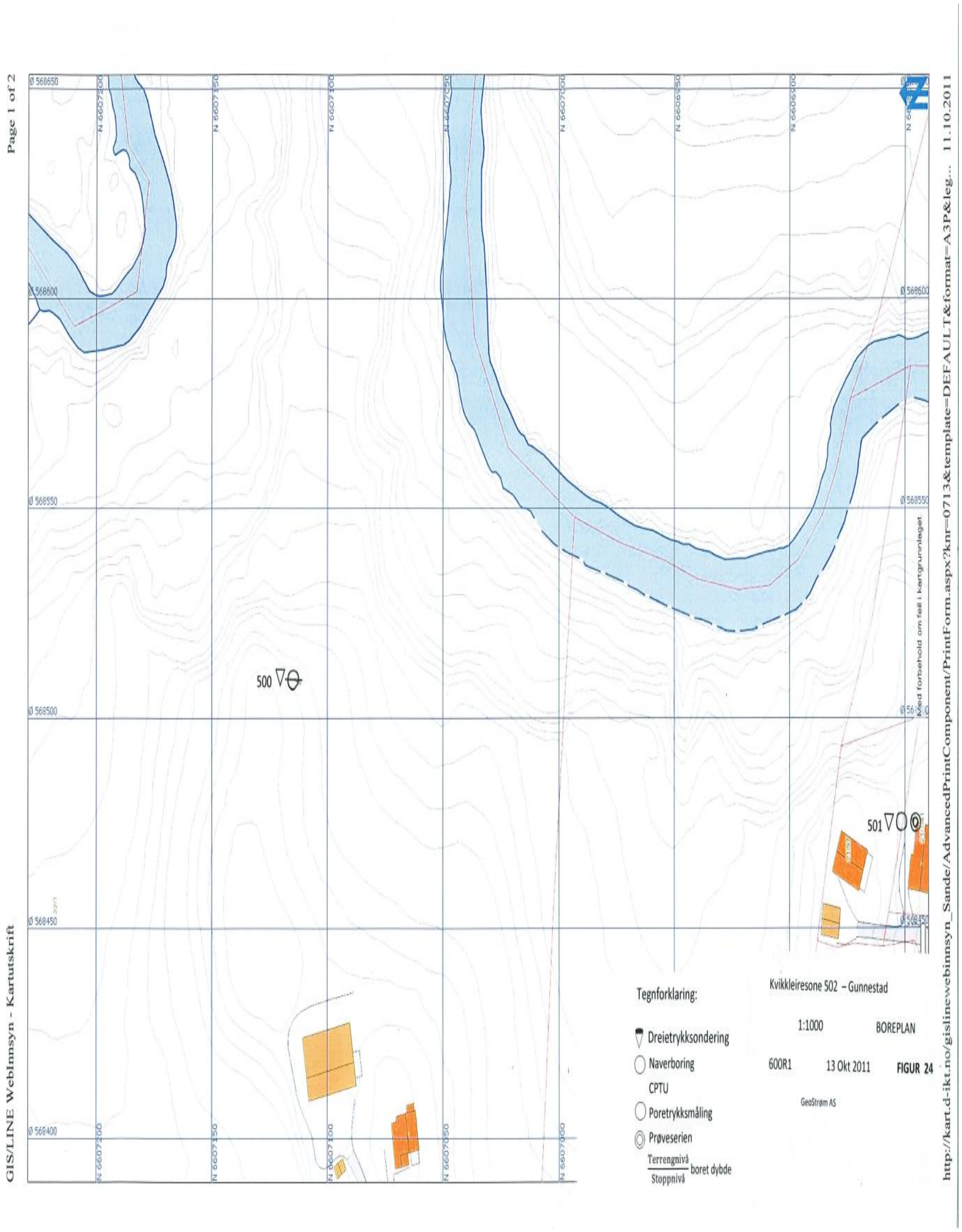
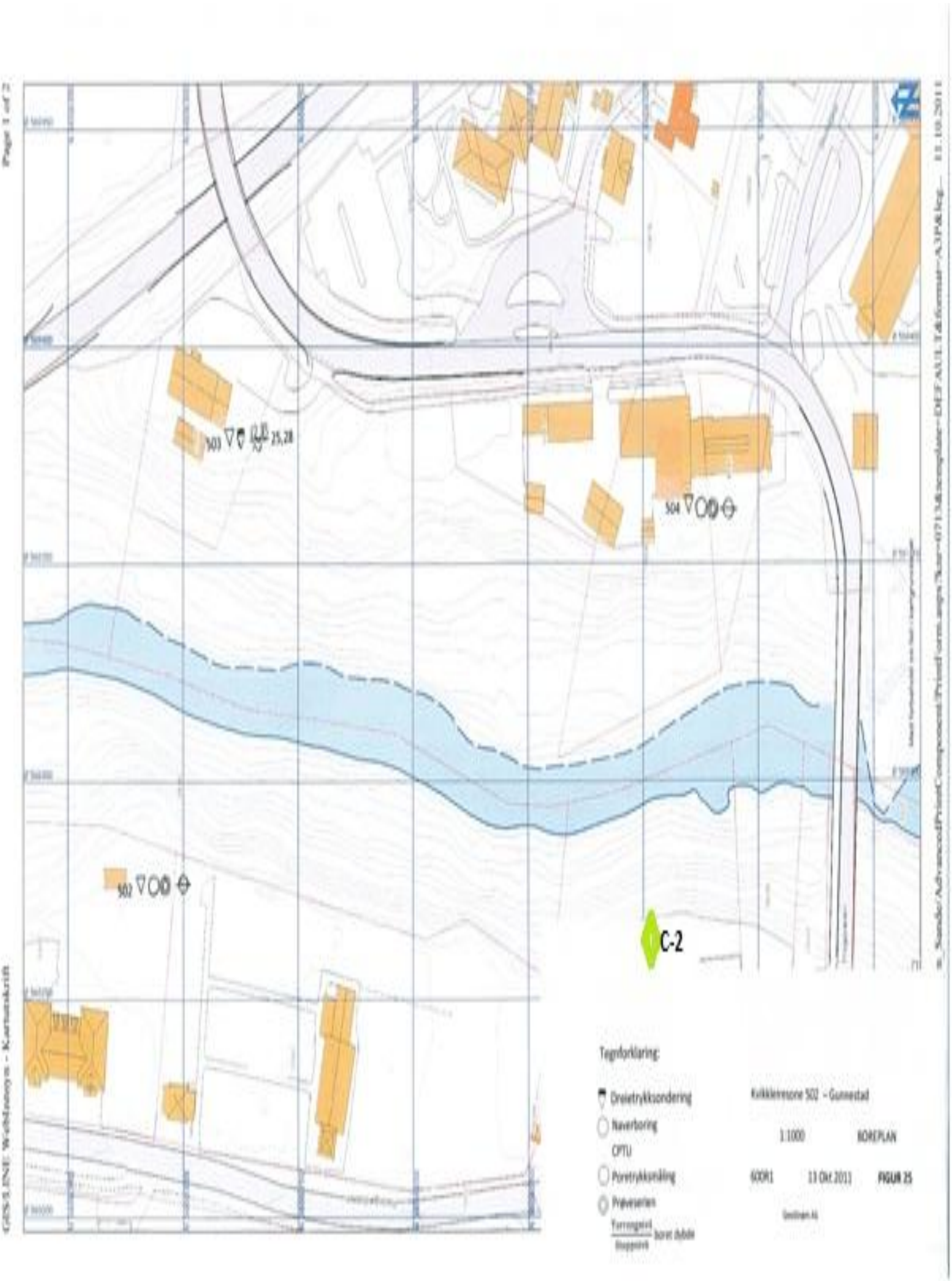
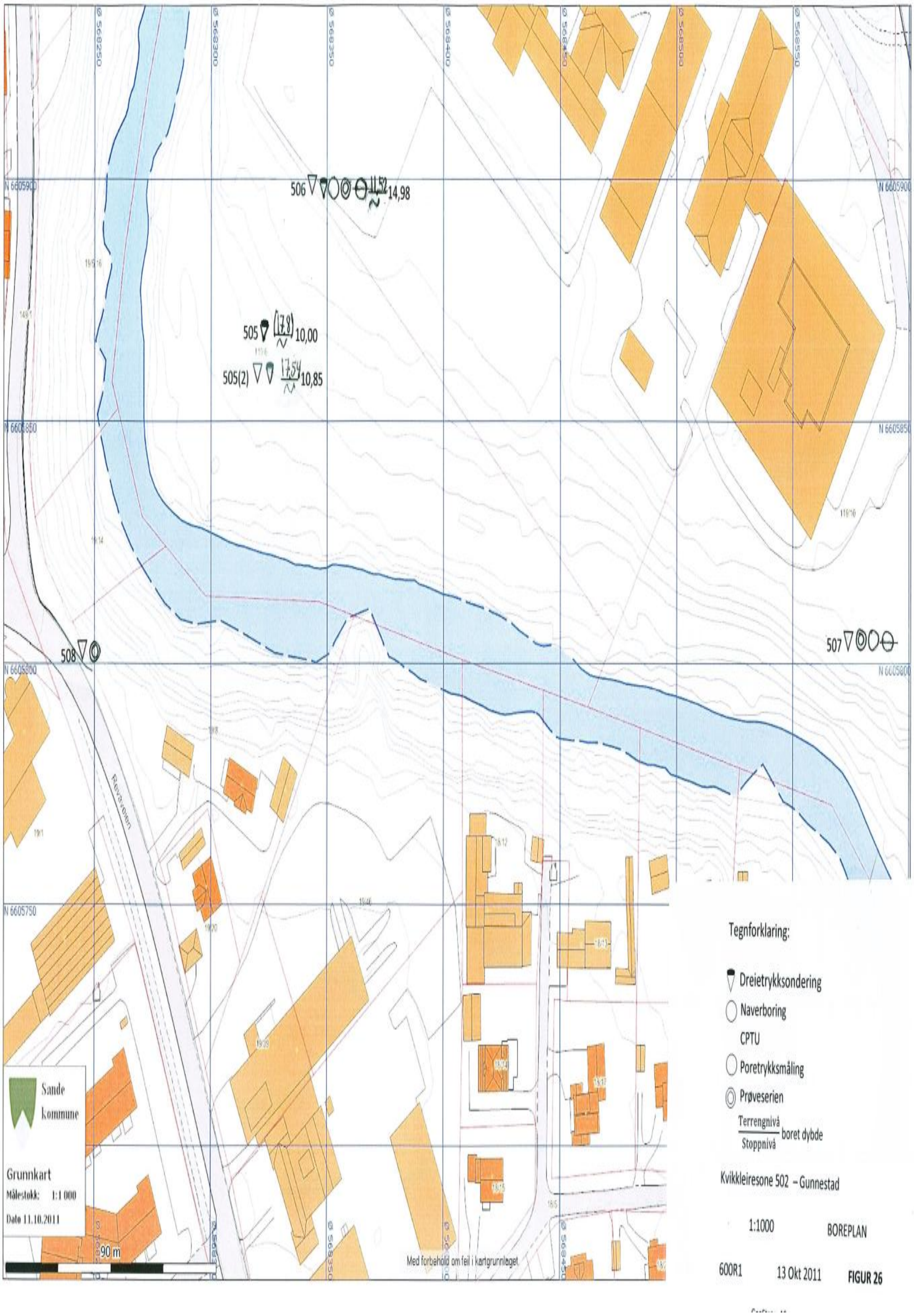
Appendices

Appendix-1 Location of CPTu soundings together with pore pressure measurements, rotary pressure sounding and sample test sites at a scale of 1:1000




503
25,28
504
502
C-2
Tegnforklaring:
Dreietrykksondering
Naverboring
CPTU
Poretrykksmåling
Prøveserien
boret dybde
Kvikkleiresone 502 – Gunnestad
1:1000
BOREPLAN
600R1
13 Okt 2011
FIGUR 25



506
14,98
505
10,00
505(2)
10,85
508
507
Revaveien
Sande kommune
Grunnkart
Målestokk: 1:1 000
Dato 11.10.2011
90 m
Med forbehold om feil i kartgrunnlaget
Tegnforklaring:
Dreietrykksondering
Naverboring
CPTU
Poretrykksmåling
Prøveserien
Terrengnivå
Stoppnivå
boret dybde
Kvikkleiresone 502 – Gunnestad
1:1000
BOREPLAN
600R1
13 Okt 2011
FIGUR 26

Appendix 2 CPT probe calibrations

GEOTECH

CERTIFICATE FOR CPT PROBE 3259

Probe No	3259	
Date of Calibration	20101117	
Replacement of		
Calibrated by	Fredric Nyström	
File name	3259 20101117 142858.doc	

Point Resistance

Maximum Load	50	MPa	
Range	50	Mpa	
Scaling Factor	**1297**		
Resolution	18.82	kPa	(12 bit resolution)
Resolution	0.5882	kPa	(18 bit resolution)
Area factor (a)	0.594		

ERRORS

Max. Temperature effect when not loaded 24.1162 kPa

Temperature range 0 –40 deg. Celsius.

Local Friction

Maximum Load	0.5	MPa	
Range	0.5	Mpa	
Scaling Factor	**6159**		
Resolution	0.20	kPa	(12 bit resolution)
Resolution	0.0062	kPa	(18 bit resolution)
Area factor (b)	0.012		

ERRORS

Max. Temperature effect when not loaded 0.5952 kPa

Temperature range 0 –40 deg. Celsius.

Pore Pressure

Maximum Load	2.5	MPa	
Range	2.5	Mpa	
Scaling Factor	**2494**		
Resolution	0.98	kPa	(12 bit resolution)
Resolution	0.0306	kPa	(18 bit resolution)

ERRORS

Max. Temperature effect when not loaded 1.8666 kPa

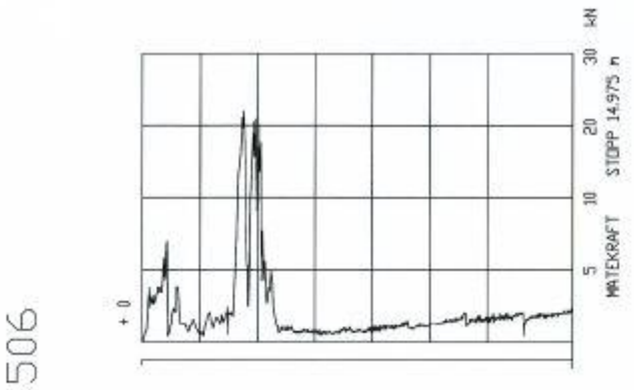
Temperature range 0 –40 deg. Celsius.

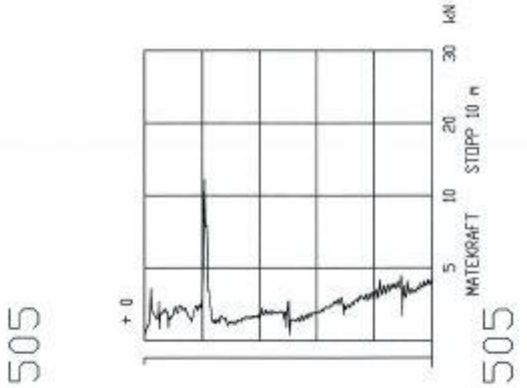
Tilt Angle

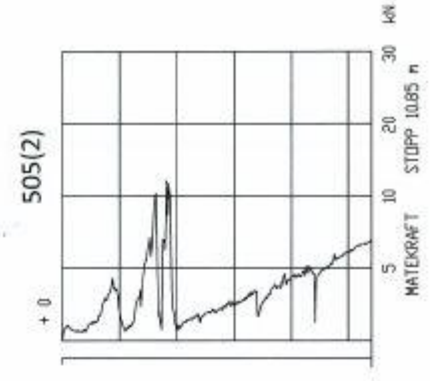
Range	0 - 40	Deg.

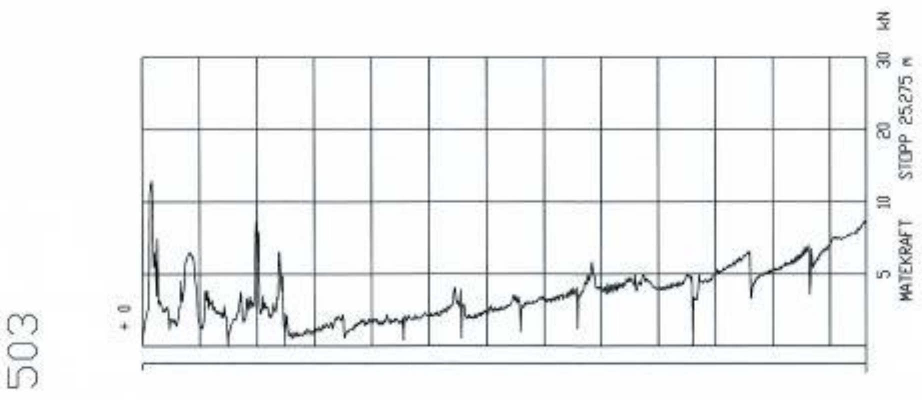
Ingenjörsfirman Geotech AB
Datavägen 53
SE-436 32 ASKIM, Sweden
+46 (0)31-28 99 20
+46 (0)31-68 16 39
www.geotech.se
VAT No.

Appendix-3 results of rotary pressure soundings.

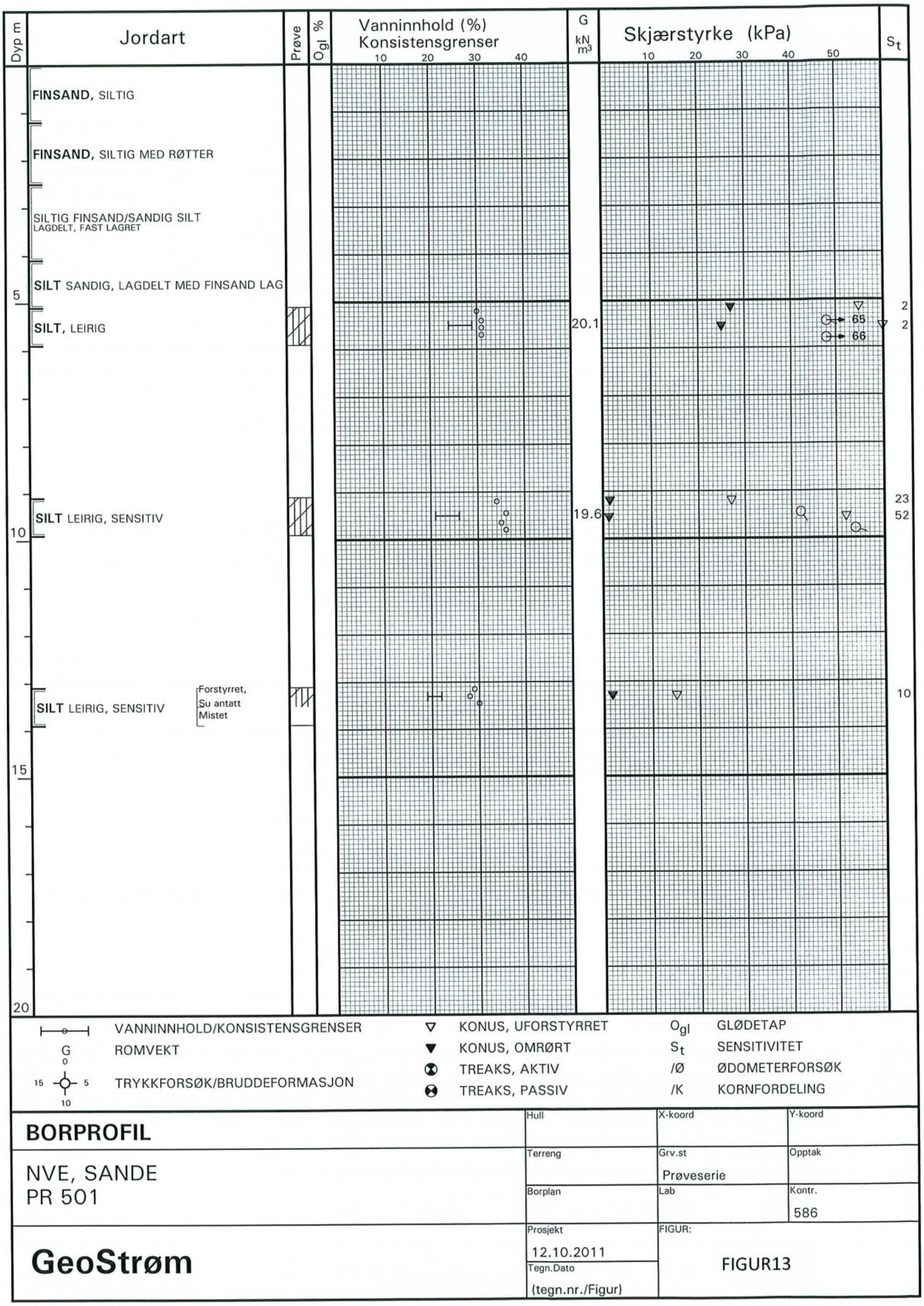


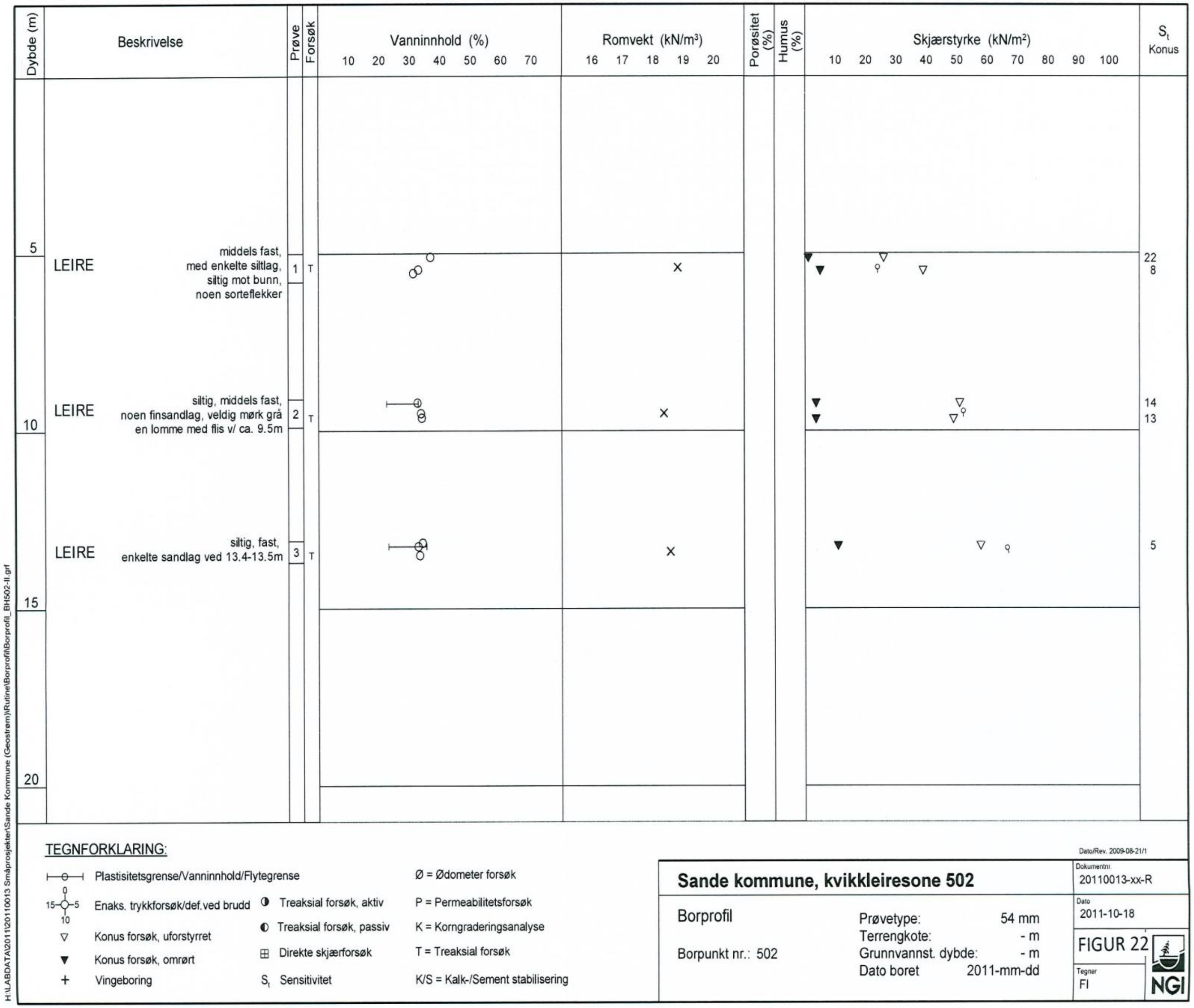


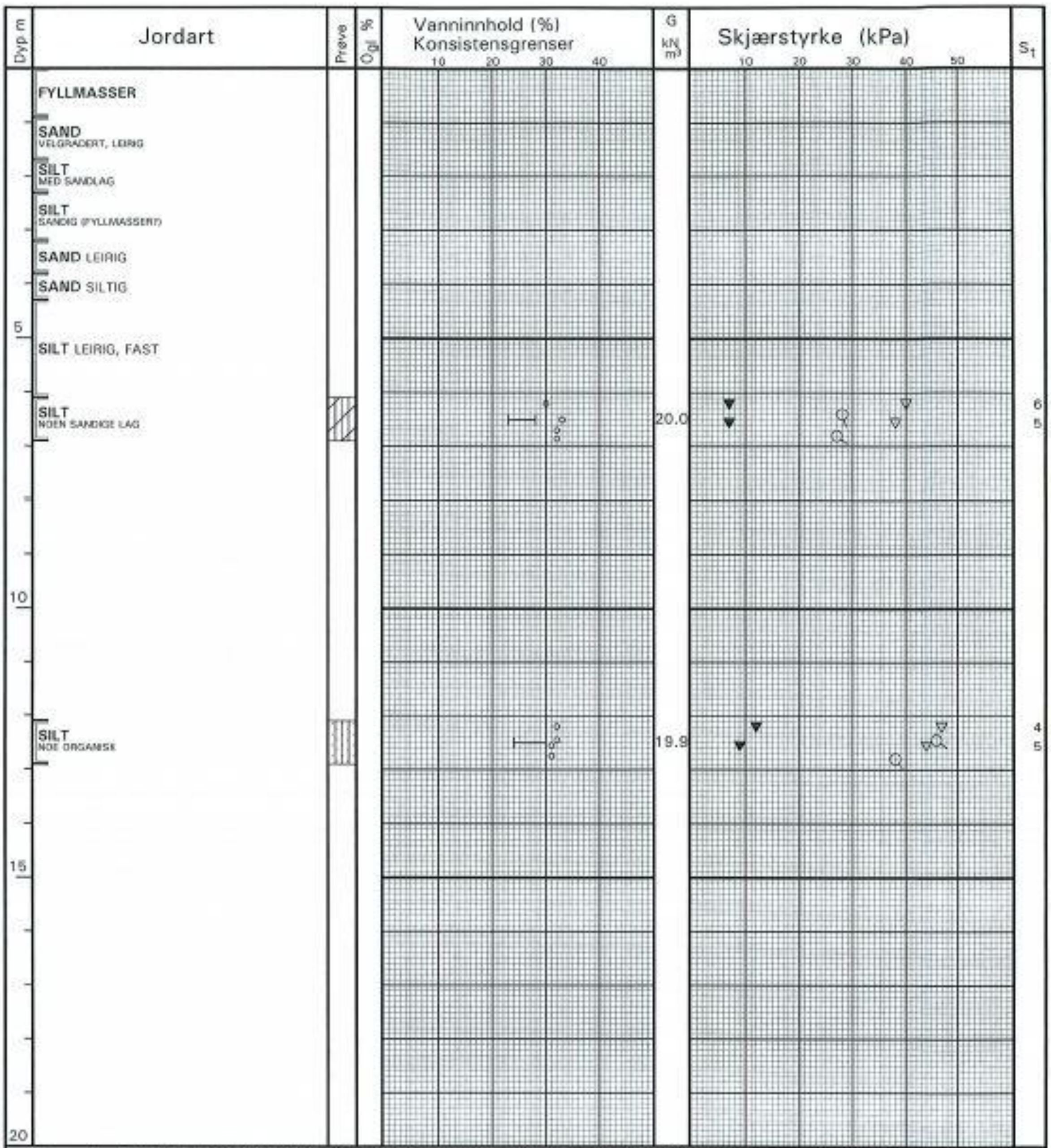




Appendix-4 Index properties and sensitivity measurements

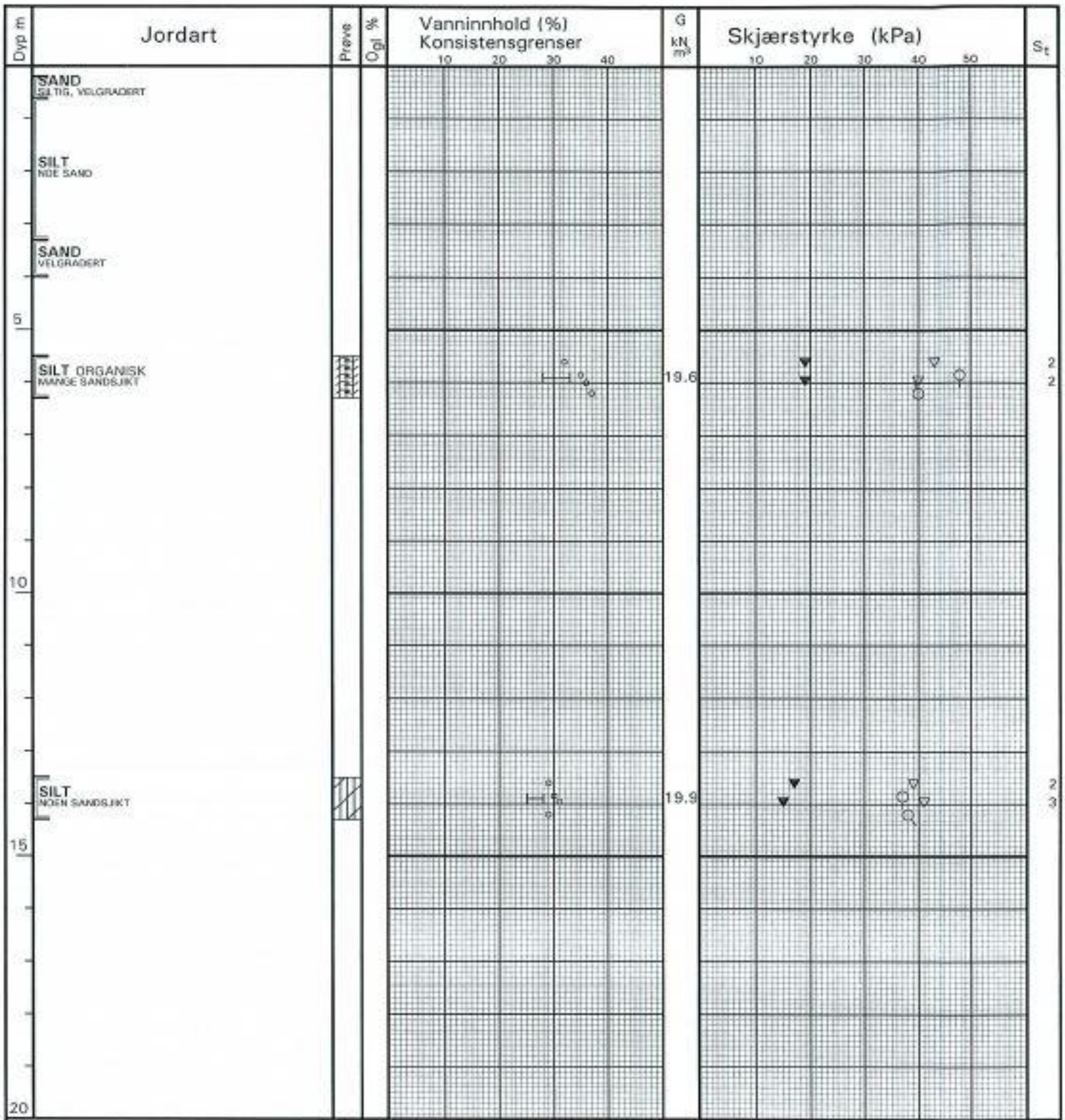



Dybde (m)
Beskrivelse
Prøve
Forsøk
Vanninnhold (%)
10 20 30 40 50 60 70
Romvekt (kN/m³)
16 17 18 19 20
Porøsitet (%)
Humus (%)
Skjærstyrke (kN/m²)
10 20 30 40 50 60 70 80 90 100
S_t Konus
5
10
15
20
LEIRE
middels fast, med enkelte siltlag, siltig mot bunn, noen sorteflekker
1 T
22
8
LEIRE
siltig, middels fast, noen finsandlag, veldig mørk grå en lomme med flis v/ ca. 9.5m
2 T
14
13
LEIRE
siltig, fast, enkelte sandlag ved 13.4-13.5m
3 T
5
TEGNFORKLARING:
Plastisitetsgrense/Vanninnhold/Flytegrense
Enaks. trykkforsøk/def.ved brudd
Konus forsøk, uforstyrret
Konus forsøk, omrørt
Vingeboring
Treaksial forsøk, aktiv
Treaksial forsøk, passiv
Direkte skjærforsøk
S_t Sensitivitet
Ø = Ødometer forsøk
P = Permeabilitetsforsøk
K = Korngraderingsanalyse
T = Treaksial forsøk
K/S = Kalk-/Sement stabilisering
Sande kommune, kvikkleiresone 502
Borprofil
Borpunkt nr.: 502
Prøvetype: 54 mm
Terrengkote: - m
Grunnvannst. dybde: - m
Dato boret 2011-mm-dd
Dato/Rev. 2009-08-21/1
Dokumentnr. 20110013-xx-R
Dato 2011-10-18
FIGUR 22
Tegner FI
NGI
H:\LABDATA\2011\20110013 Småprosjekter\Sande Kommune (Geostrøm)\Rutine\Borprofil\Borprofil_BH502-II.grf



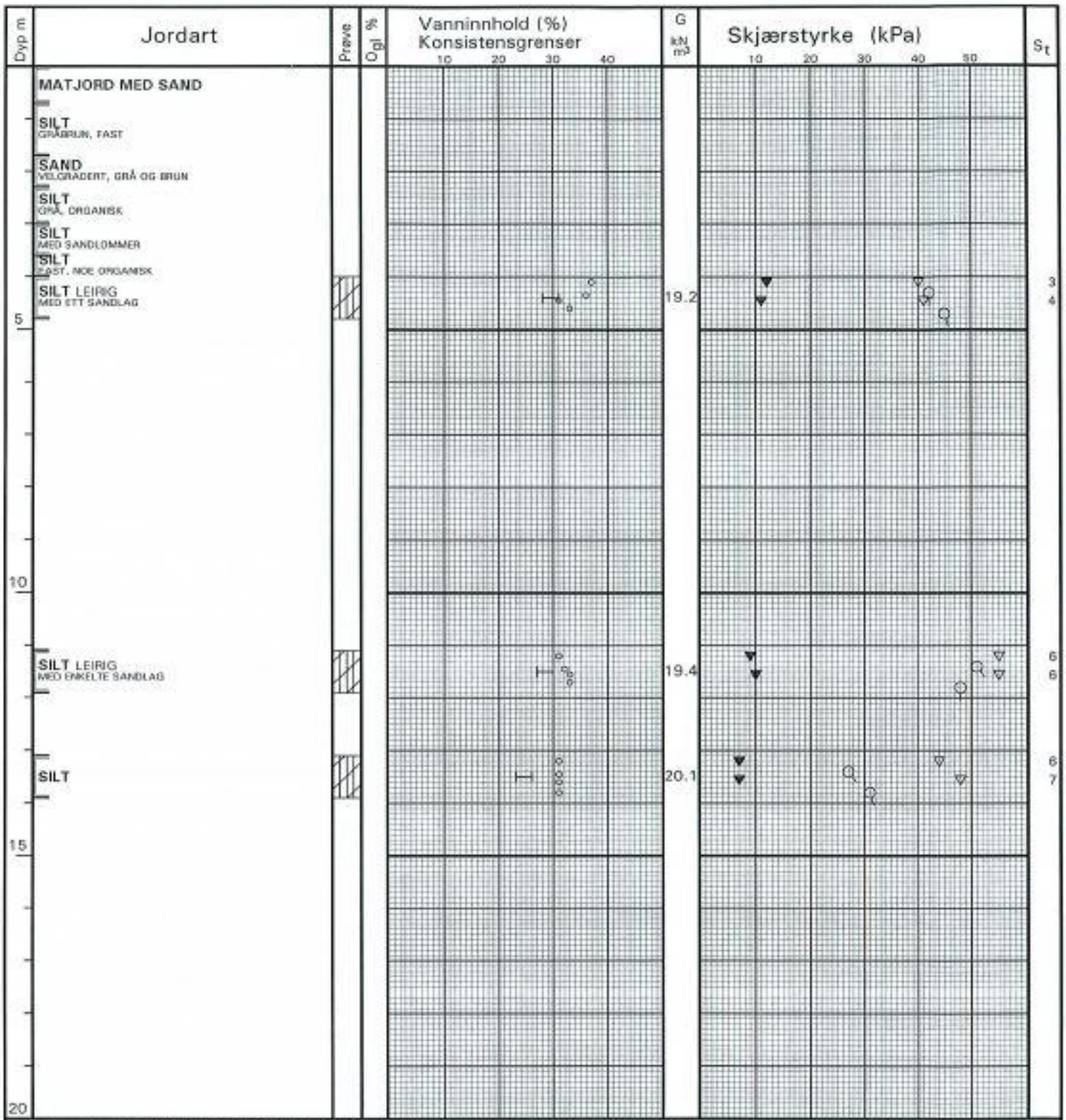
VANNINNHOLD/KONSISTENSGRENSER	▽ KONUS, UFORSTYRRET	O_{gl}	GLØDETAP
G ROMVEKT	▼ KONUS, OMRØRT	S_t	SENSITIVITET
TRYKKFORSØK/BRUDDEFORMASJON	TREAKS, AKTIV	/Ø	ØDOMETERFORSØK
	TREAKS, PASSIV	/K	KORNFORDELING

BORPROFIL	Hull: PR504	X-koord	Y-koord
NVE SANDE	Terreng	Grv.st	Opptak: Prøveserie
	Borplan	Lab	Kontr.
GeoStrøm	Prosjekt: 600 Tegn.Dato: 6.9.2011	FIGUR: **Fig**	



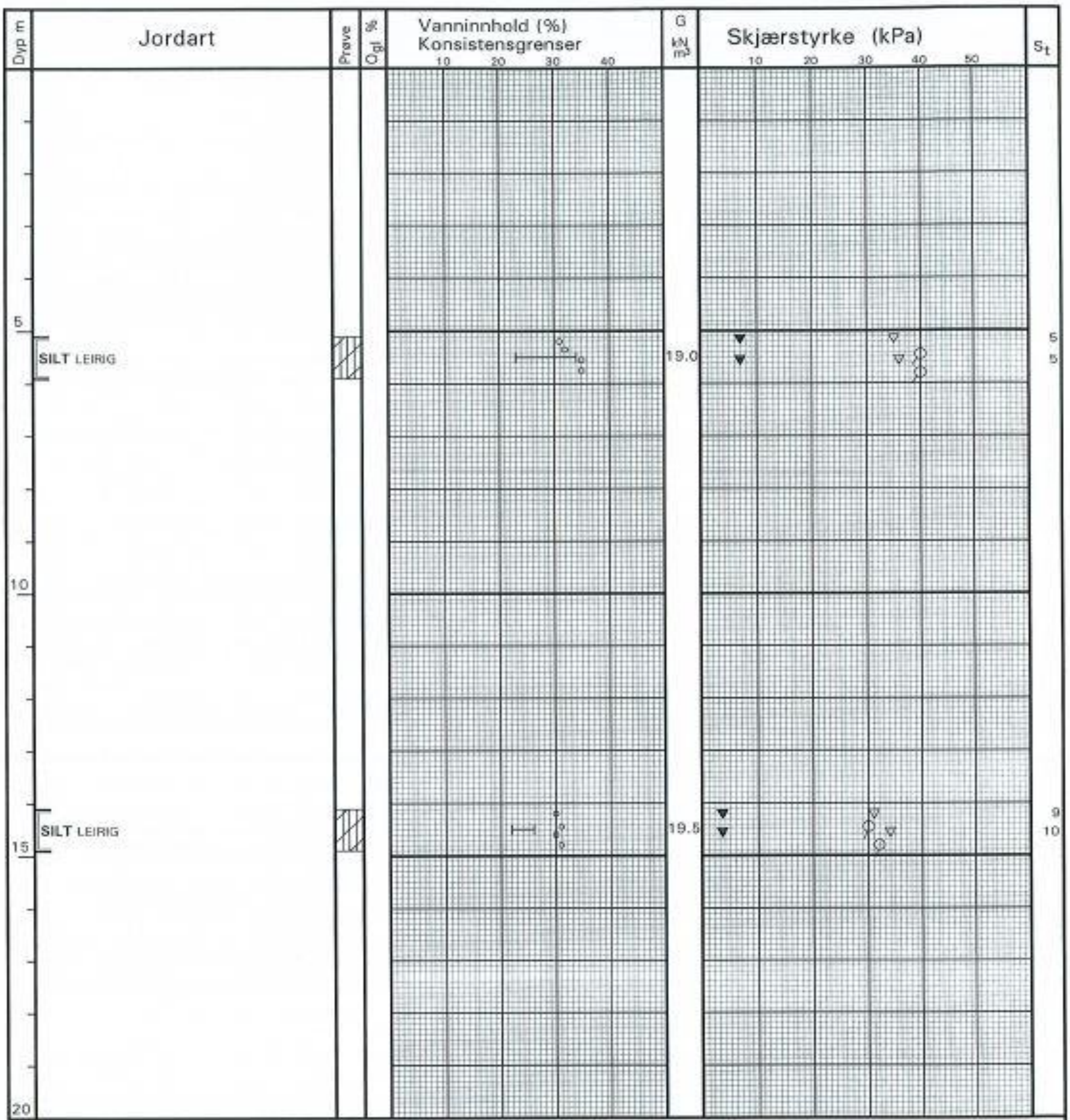
	VANNINNHOLD/KONSISTENSGRENSER	▽	KONUS, UFORSTYRRET	O_{gl}	GLØDETAP
G	ROMVEKT	▼	KONUS, OMRØRT	S_t	SENSITIVITET
	TRYKKFORSØK/BRUDDEFORMASJON		TREAKS, AKTIV	/Ø	ØDOMETERFORSØK
			TREAKS, PASSIV	/K	KORNFORDELING

BORPROFIL	Hull: PR506	X-koord	Y-koord
NVE SANDE	Terreng	Grv.st	Opptak: Prøveserie
	Borplan	Lab	Kontr.
GeoStrøm	Prosjekt: 600	FIGUR: Fig	
	Tegn.Dato: 6.9.2011		



	VANNINNHOLD/KONSISTENSGRENSER	▽	KONUS, UFORSTYRRET	O_{gl}	GLØDETAP
G	ROMVEKT	▼	KONUS, OMRØRT	S_t	SENSITIVITET
	TRYKKFORSØK/BRUDDEFORMASJON		TREAKS, AKTIV	/Ø	ØDOMETERFORSØK
			TREAKS, PASSIV	/K	KORNFORDELING

BORPROFIL	Hull: PR507	X-koord	Y-koord
NVE SANDE	Terreng	Grv.st	Opptak: Prøveserie
	Borplan	Lab	Kontr.
GeoStrøm	Prosjekt: 600	FIGUR: **Fig**	
	Tegn.Dato: 6.9.2011		



VANNINNHOLD/KONSISTENSGRENSER	▽ KONUS, UFORSTYRRET	O_{gl}	GLØDETAP
G ROMVEKT	▼ KONUS, OMRØRT	S_t	SENSITIVITET
TRYKKFORSØK/BRUDDEFORMASJON	TREAKS, AKTIV	/Ø	ØDOMETERFORSØK
	TREAKS, PASSIV	/K	KORNFORDELING

BORPROFIL	Hull: 508	X-koord	Y-koord
NVE, SANDE PR 508	Terreng	Grv.st: Prøveserie	Opptak
	Borplan	Lab	Kontr.: 600
GeoStrøm	Prosjekt: 22.10.2011 Tegn.Dato: (tegn.nr./Figur)	FIGUR: FIGUR 17	

Appendix-5 Borehole profile from auger drilling

PROSJEKT : 600	NAVERBORINGER	FIGUR: 18
Sande		DATO: 6 juni 2011

BORING: 501			Vannstand
DYP	**W**	**Lab. beskrivelse**	**Markbeskrivelse**
			Grå og brun siltig finsand med rust-røde flekker
- 0,5			
- 1,0			Gråbrun siltig fin-sand med røtter, meget fast
- 1,5			
- 2,0			
- 2,5			Grå siltig finsand/ sandig silt med rustrøde flekker, meget fast, lagdelt
- 3,0			
- 3,5			
- 4,0			Grå sandig silt, lagdelt med lysere lag av finsand middels fast
- 4,5			
- 5,0			
- 5,5			Hylse 221
- 6,0			
- 6,5			

BORING: 501			
DYP	**W**	**Lab. beskrivelse**	**Markbeskrivelse**
- 7,0			
- 7,5			
- 8,0			
- 8,5			
- 9,0			
- 9,5			Hylse 432
- 10,0			Grå siltig leire
- 10,5			
- 11,0			
- 11,5			
- 12,0			
- 12,5			
- 13,0			Hylse 592 Mistet nederste 0,2 meter.

W er vann i % av tørr vekt.

Prøver fra naverboringer vil være forstyrret og derfor bløtere enn uforstyrret grunn. Lagdeling kan bli borte. Laboratoriebeskrivelsene må derfor brukes sammen med markbeskrivelsene.

PROSJEKT : 600	NAVERBORINGER	FIGUR: 18
Sande		DATO: 6 juni 2011

BORING: 501			Vannstand
DYP	W	Lab. beskrivelse	Markbeskrivelse
			Grå og brun siltig finsand med rust-røde flekker
- 0,5			
- 1,0			Gråbrun siltig fin-sand med røtter, meget fast
- 1,5			
- 2,0			
- 2,5			Grå siltig finsand/ sandig silt med rustrøde flekker, meget fast, lagdelt
- 3,0			
- 3,5			
- 4,0			Grå sandig silt, lagdelt med lysere lag av finsand middels fast
- 4,5			
- 5,0			
- 5,5			Hylse 221
- 6,0			
- 6,5			

BORING: 501			
DYP	W	Lab. beskrivelse	Markbeskrivelse
- 7,0			
- 7,5			
- 8,0			
- 8,5			
- 9,0			
- 9,5			Hylse 432
- 10,0			Grå siltig leire
- 10,5			
- 11,0			
- 11,5			
- 12,0			
- 12,5			
- 13,0			Hylse 592 Mistet nederste 0,2 meter.

W er vann i % av tørr vekt.

Prøver fra naverboringer vil være forstyrret og derfor bløtere enn uforstyrret grunn. Lagdeling kan bli borte. Laboratoriebeskrivelsene må derfor brukes sammen med markbeskrivelsene.

PROSJEKT : 600		FIGUR: 20
NVE - Sande	**NAVERBORING**	DATO: 5.7.11

BORING: 506

DYP	W	Lab. beskrivelse	Markbeskrivelse
			matjord
			Siltig velgradert sand
- 0,5			
			Gråbrun fast silt med noe sand og oksidasjonsflekker
- 1,0			
- 1,5			
- 2,0			
- 2,5			
- 3,0			
			Velgradert sand, brun med rustrøde Partier
- 3,5			
- 4,0			Grå silt med brune flekker
- 4,5			
- 5,0			
- 5,5			Fortsatt prøveserie
- 6,0			
- 6,5			

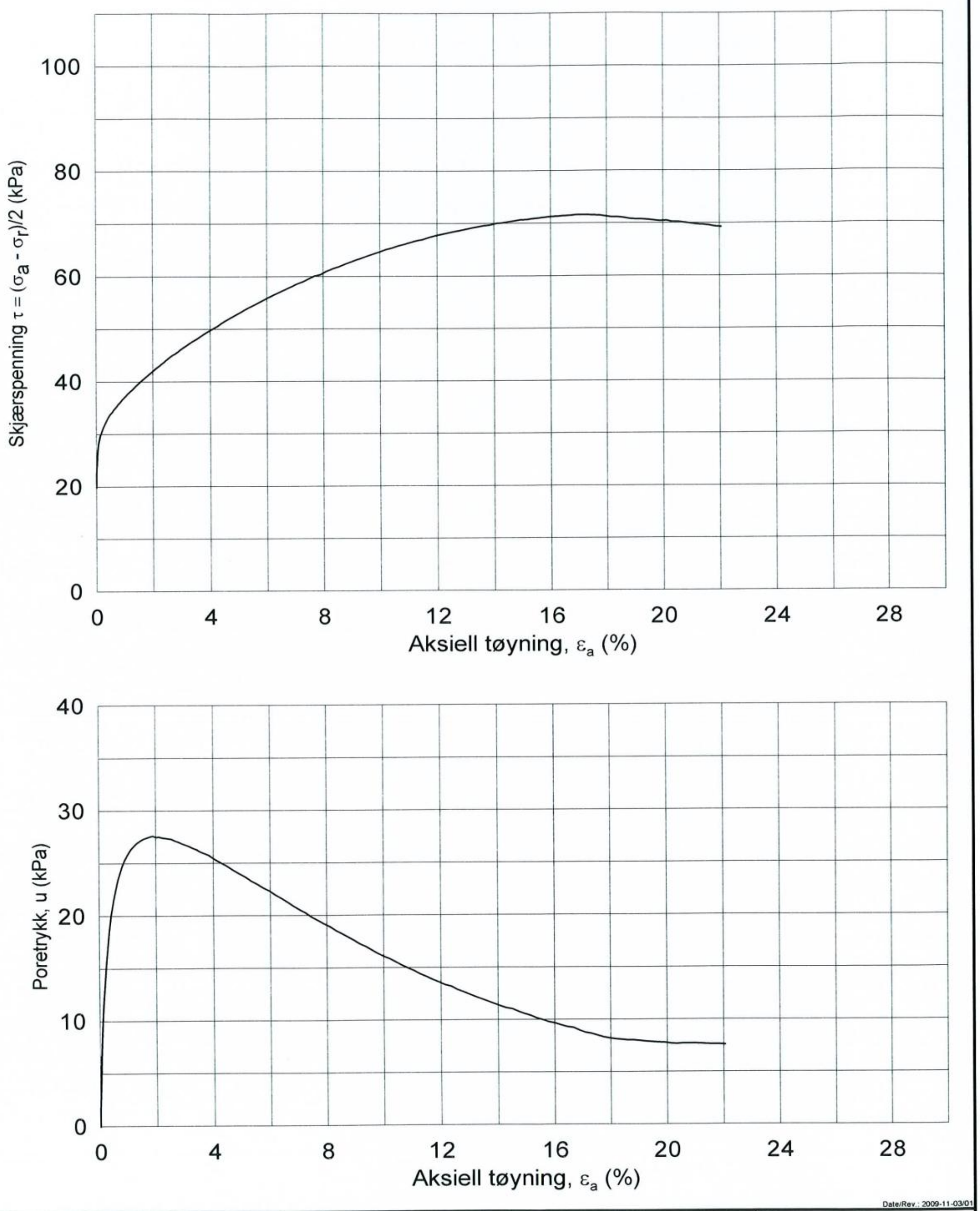
BORING:

DYP	W	Lab. beskrivelse	Markbeskrivelse
- 0,5			
- 1,0			
- 1,5			
- 2,0			
- 2,5			
- 3,0			
- 3,5			
- 4,0			
- 4,5			
- 5,0			
- 5,5			
- 6,0			
- 6,5			

W er vann i % av tørr vekt.

Prøver fra naverboringer vil være forstyrret og derfor bløtere enn uforstyrret grunn. Lagdeling kan bli borte. Laboratoriebeskrivelsene må derfor brukes sammen med markbeskrivelsene.

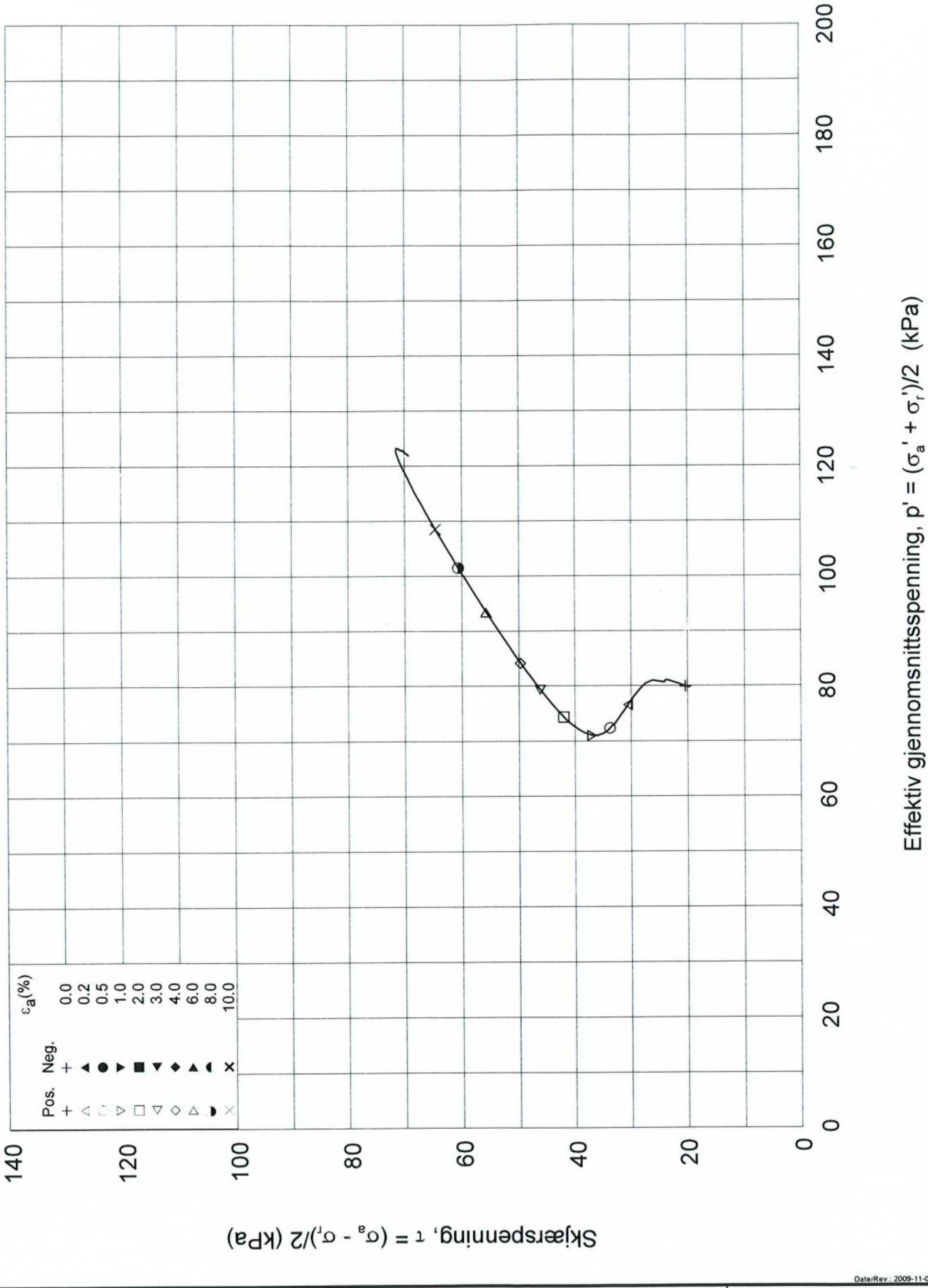
Appendix-6 Laboratory results from CAUC Triaxial tests and Oedometer tests



Date/Rev.: 2009-11-03/01

Sande Kommune, Kvikkleiresone 502			Dokument nr. 20110013-1
Treaksial forsøk: **CAUA**	Leire		Dato 2011-10-24
Boring: **502**	Dybde = **5.60** m	Konsolidering-spenninger	Figur nr. 23a
Sylinder: **1**	p_o' = **101.5** kPa	(kPa) maks. min. endelig	
Del: **A**	w_i = **30.8** %	σ_{ac}' = - - **99.5**	Tegnet av MAS
Test: **1**	w_c = **28.8** %	σ_{rc}' = - - **59.8**	NGI

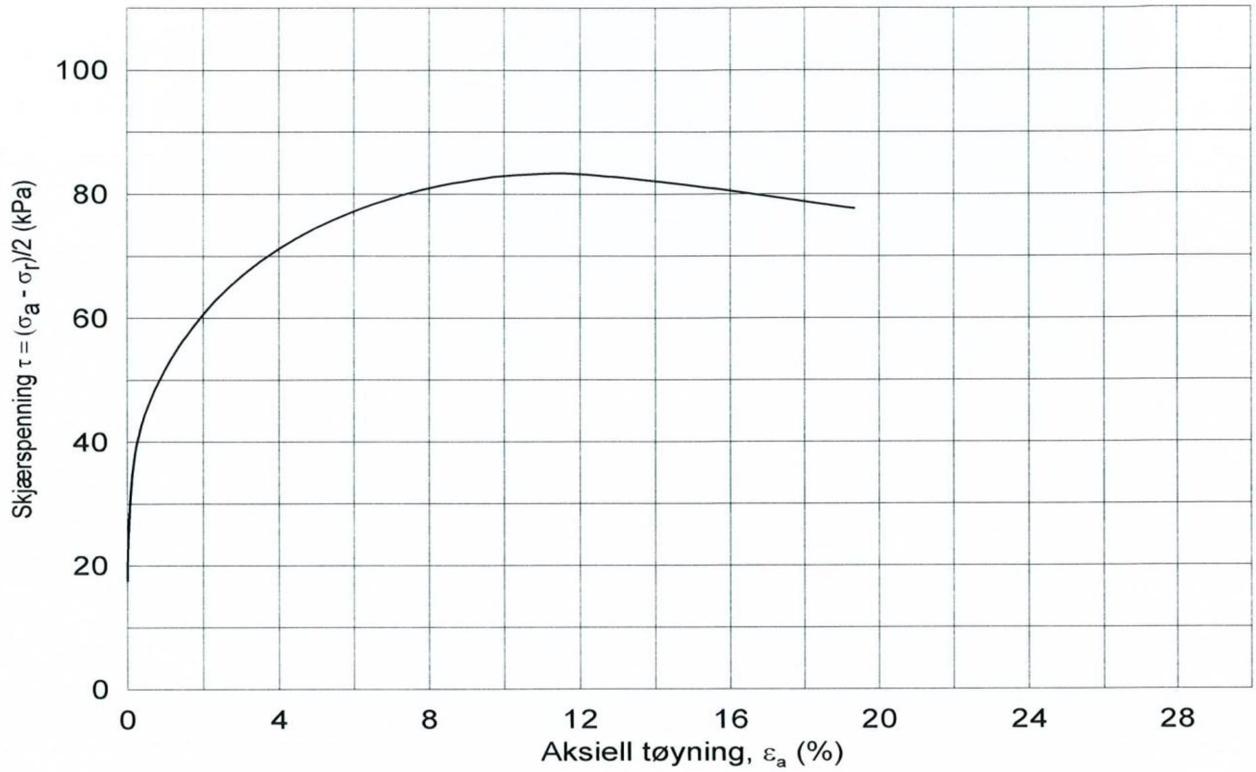
502-1-A-1.Plot1.grf

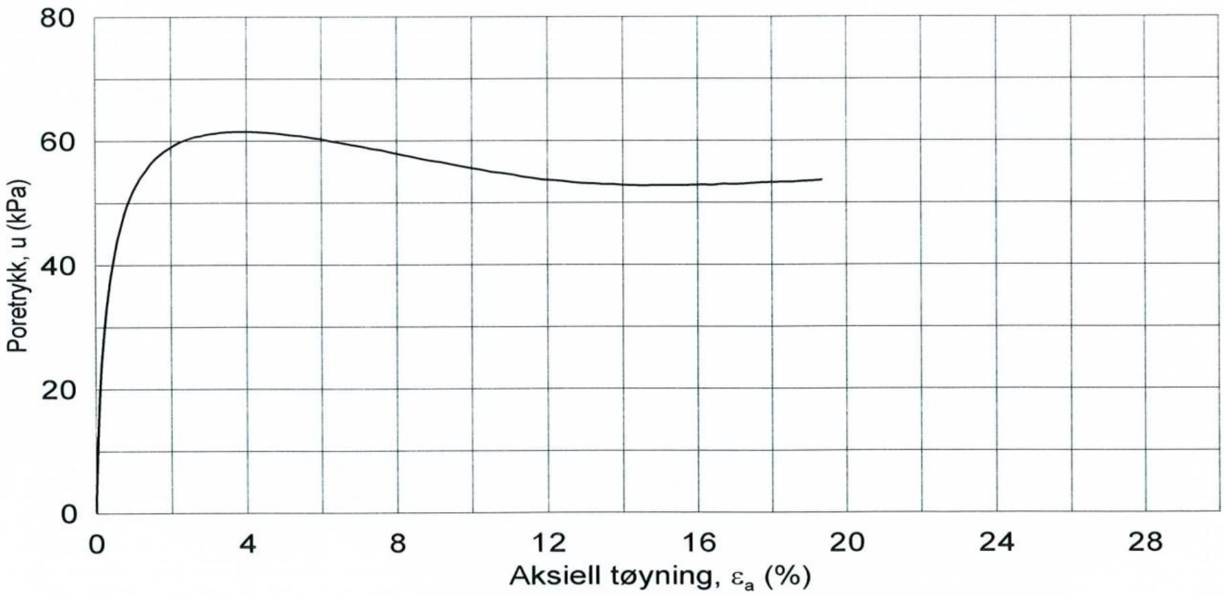


Date/Rev.: 2009-11-03/01

Sande Kommune, Kvikkleiresone 502				Dokument nr. 20110013-1
Treaksial forsøk: **CAUA**	Leire			Dato 2011-10-24
Boring: **502**	Dybde = **5.60** m	Konsolidering-spenninger		Figur nr. 23b
Sylinder: **1**	p_o' = **101.5** kPa	(kPa)	maks. min. endelig	
Del: **A**	w_i = **30.8** %	σ_{ac}' =	- - **99.5**	Tegnet av MAS
Test: **1**	w_c = **28.8** %	σ_{rc}' =	- - **59.8**	NGI

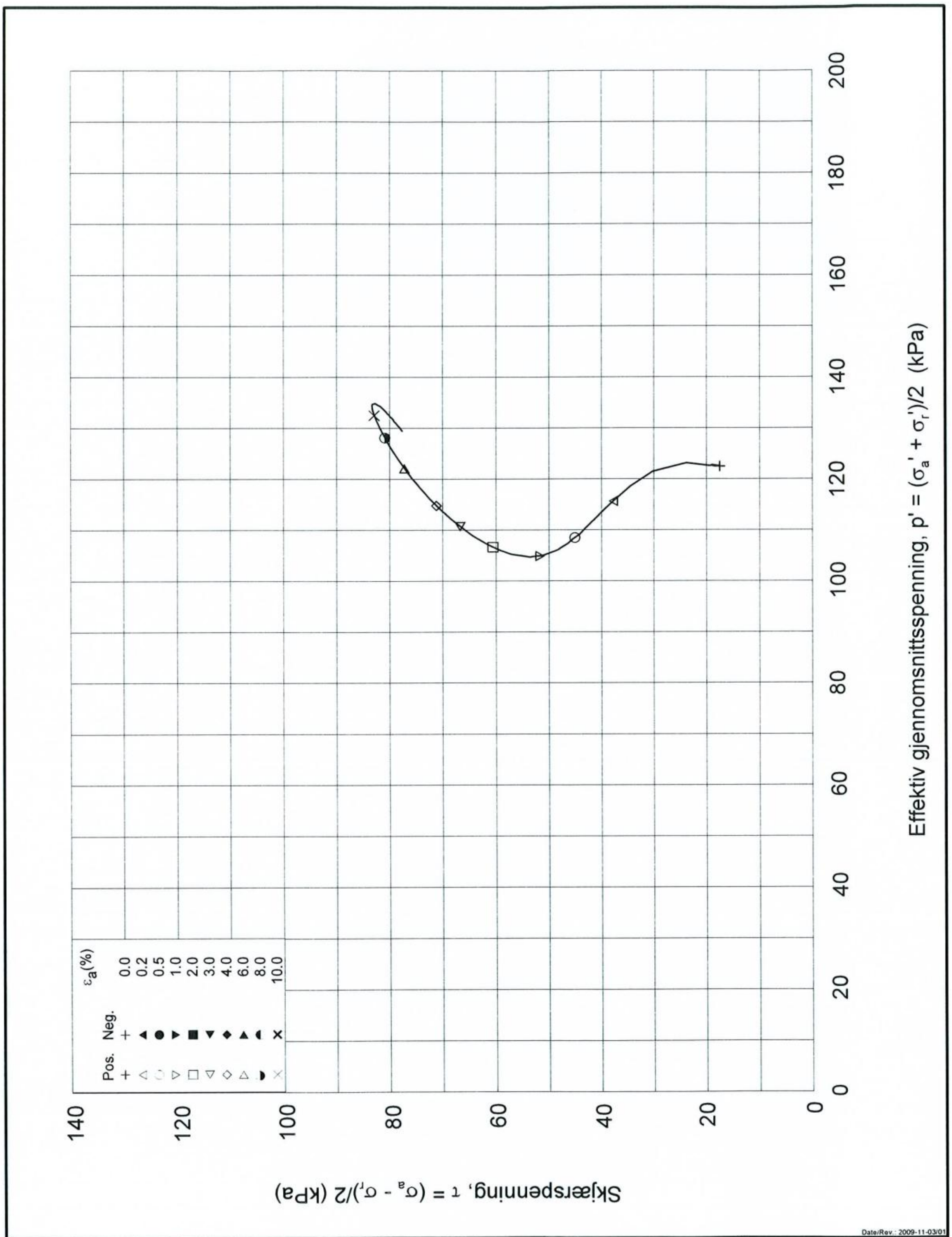
502-1-A-1.Plot2.grf



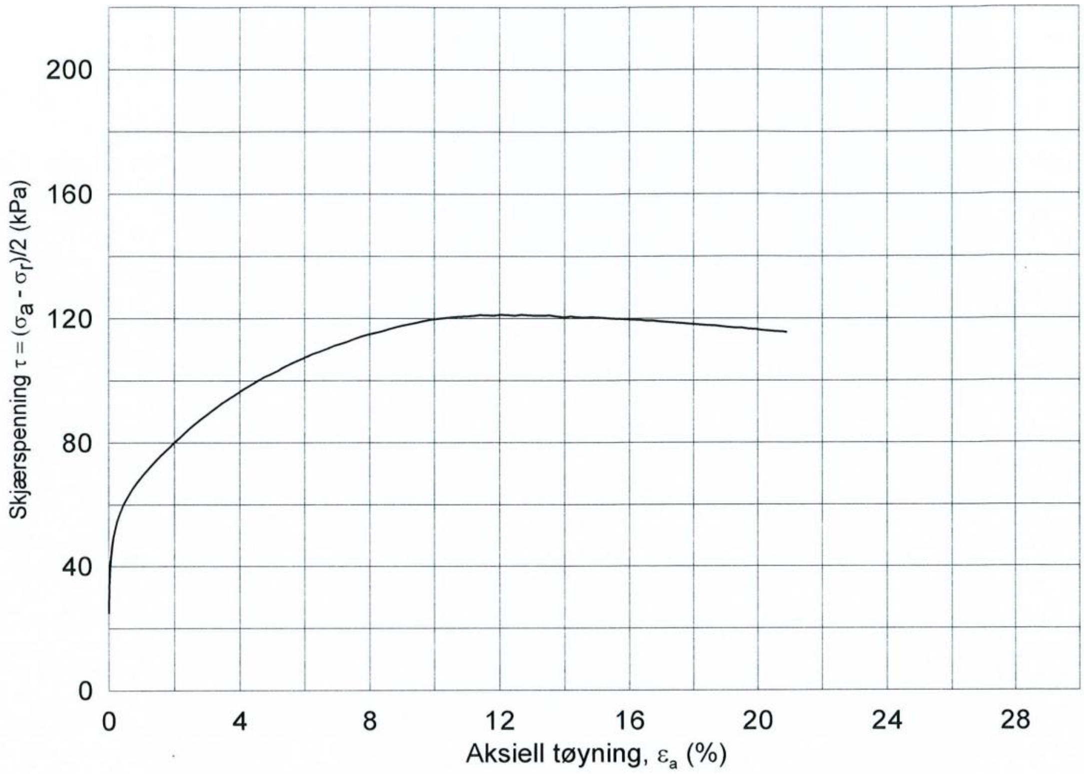


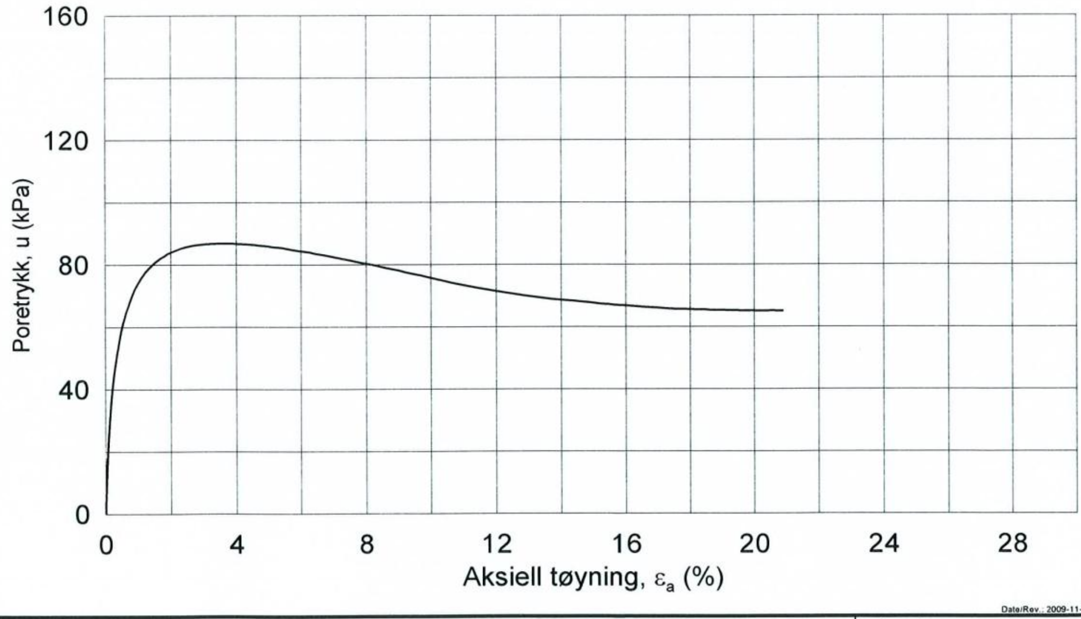
Date/Rev.: 2009-11-03/01

Sande Kommune, Kvikkleiresone 502								Dokument nr. 20110013-1
Treaksial forsøk: **CAUA**	Leire							Dato 2011-10-24
Boring: **502**	Dybde =	**9.43**	m	Konsolidering-spenninger				Figur nr. 23c
Sylinder: **2**	p_o' =	**140.0**	kPa	(kPa)	maks.	min.	endelig	
Del: **A**	w_i =	**34.7**	%	σ_{ac}' =	-	-	**140.0**	Tegnet av MAS
Test: **1**	w_c =	**32.2**	%	σ_{rc}' =	-	-	**105.0**	NGI



Sande Kommune, Kvikkleiresone 502			Dokument nr. 20110013-1	
Treaksial forsøk: **CAUA**	Leire		Dato 2011-10-24	
Boring: **502**	Dybde = **9.43** m	Konsolidering-spenninger	Figur nr. 23d	NGI
Sylinder: **2**	p_o' = **140.0** kPa	(kPa) maks. min. endelig		
Del: **A**	w_i = **34.7** %	σ_{ac}' = - - **140.0**	Tegnet av MAS	
Test: **1**	w_c = **32.2** %	σ_{rc}' = - - **105.0**		





Date/Rev.: 2009-11-03/01

Sande Kommune, Kvikkleiresone 502

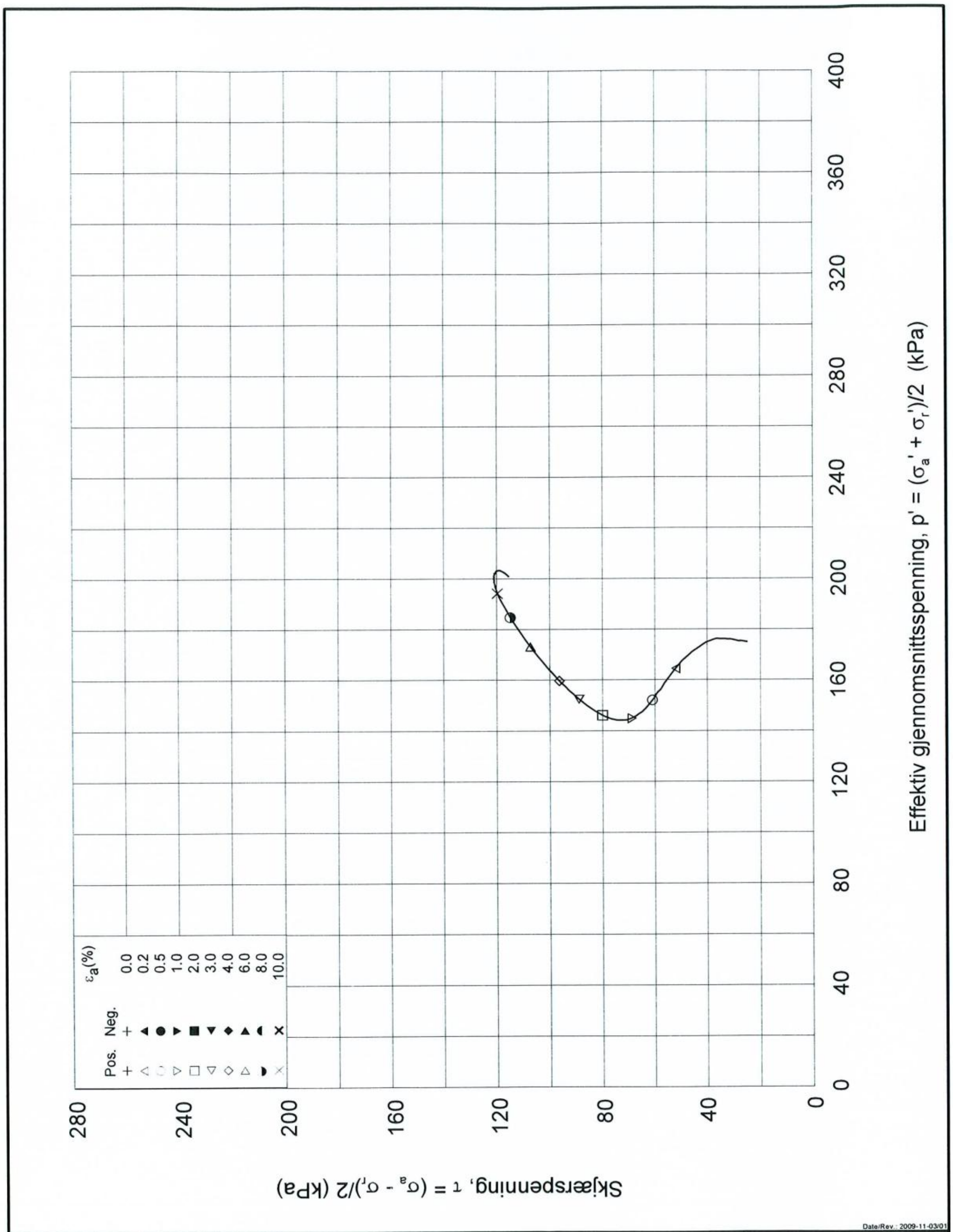
Dokument nr. 20110013-1

Dato 2011-10-24

Treaksial forsøk: **CAUA** Leire

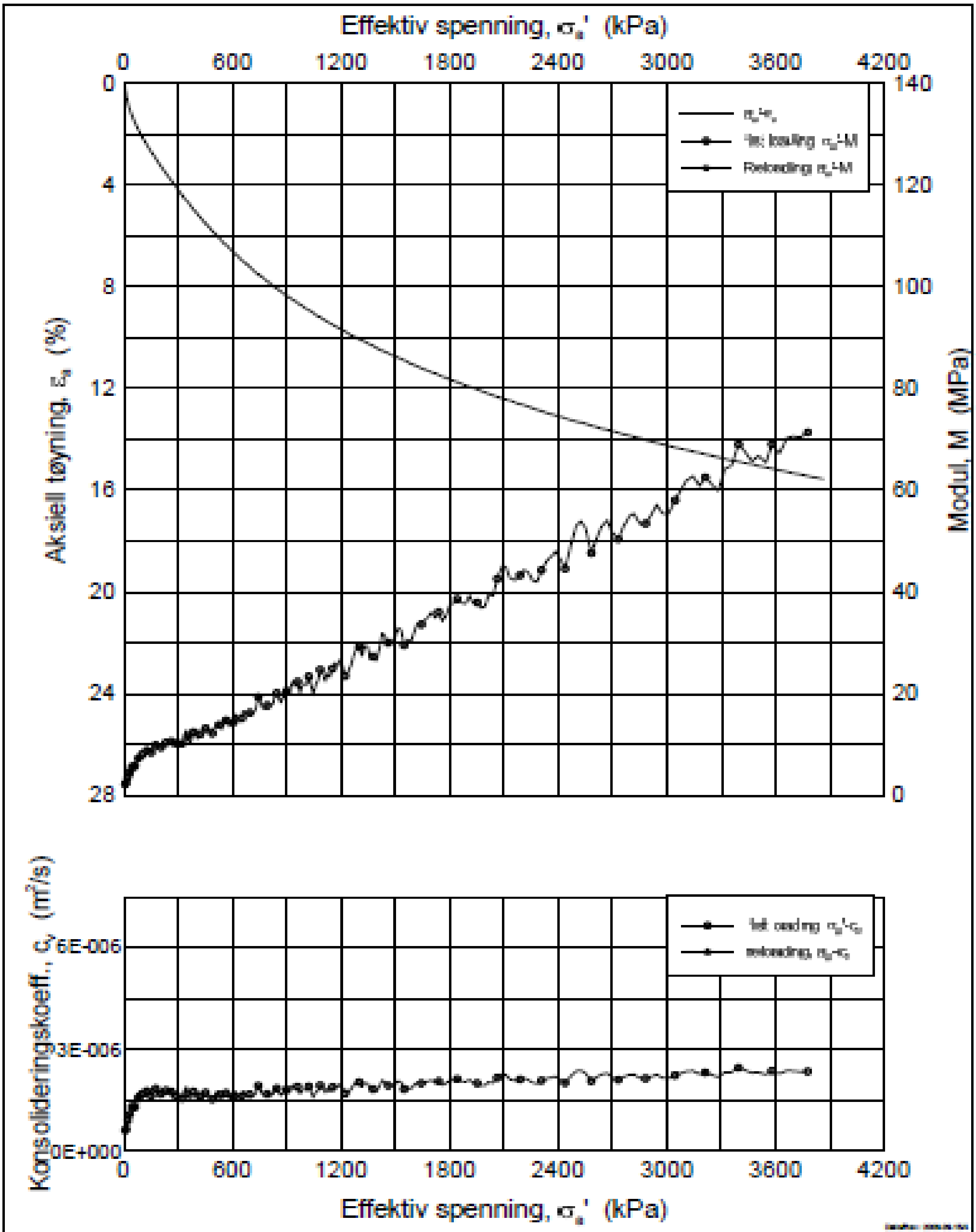
Boring:	**502**	Dybde =	**13.30**	m	Konsolidering-spenninger					Figur nr.	23e
Sylinder:	**3**	p_0' =	**200.0**	kPa	(kPa)		maks.	min.	endelig		
Del:	**A**	w_i =	**32.4**	%	σ_{ac}' =		-	-	**199.9**	Tegnet av	MAS
Test:	**1**	w_c =	**29.6**	%	σ_{rc}' =		-	-	**150.0**		

NGI



Sande Kommune, Kvikkleiresone 502			Dokument nr. 20110013-1
Treaksial forsøk: **CAUA**	Leire		Dato 2011-10-24
Boring: **502**	Dybde = **13.30** m	Konsolidering-spenninger	Figur nr. 23f
Sylinder: **3**	p_o' = **200.0** kPa	(kPa) maks. min. endelig	
Del: **A**	w_i = **32.4** %	σ_{ac}' = - - **199.9**	Tegnet av MAS
Test: **1**	w_c = **29.6** %	σ_{rc}' = - - **150.0**	NGI

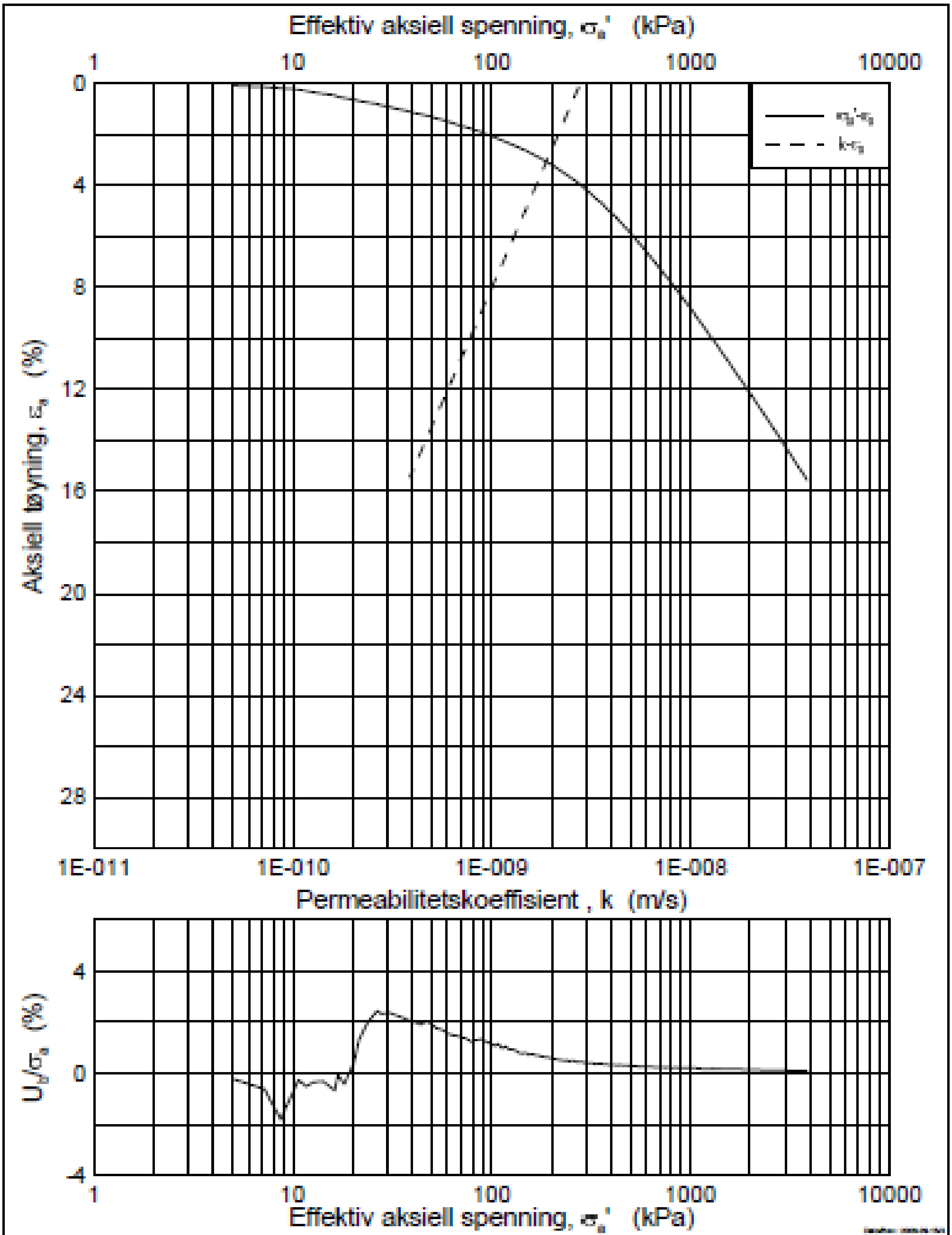
502-3-A-1.Plot2.grf



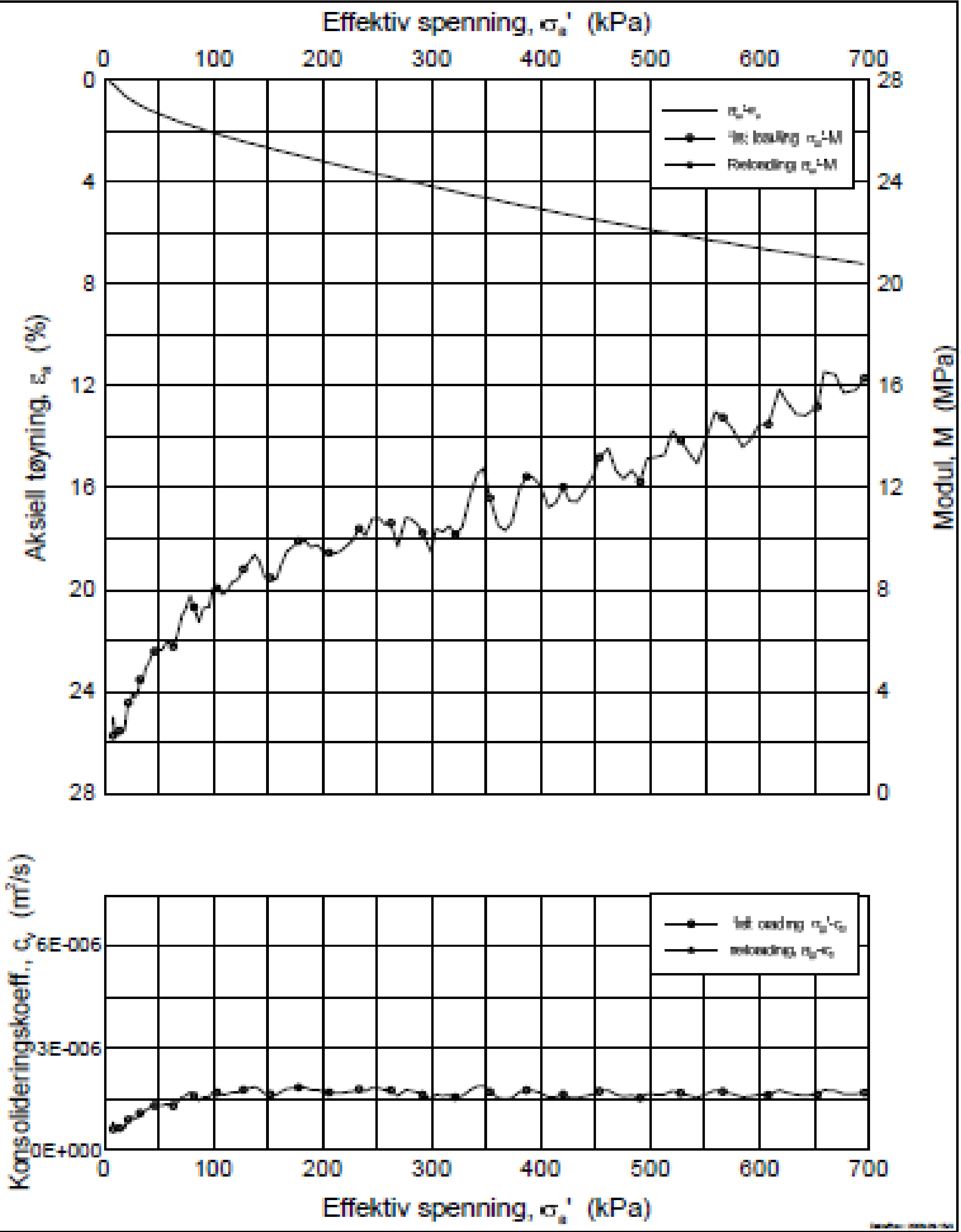
Sande kommune, kvikkleiresone 502

Oedometer test (CRSC)		Dybde = 9.30 m	Dokumentnr. 20110013
Borhull: 502	Sylinder: 2	p_0' = 93.0 kPa	Dato 2011-11-01
Del: A	Test: 1	w_i = 32.32 %	Figurnr. XX.XX
			Tegner FPf

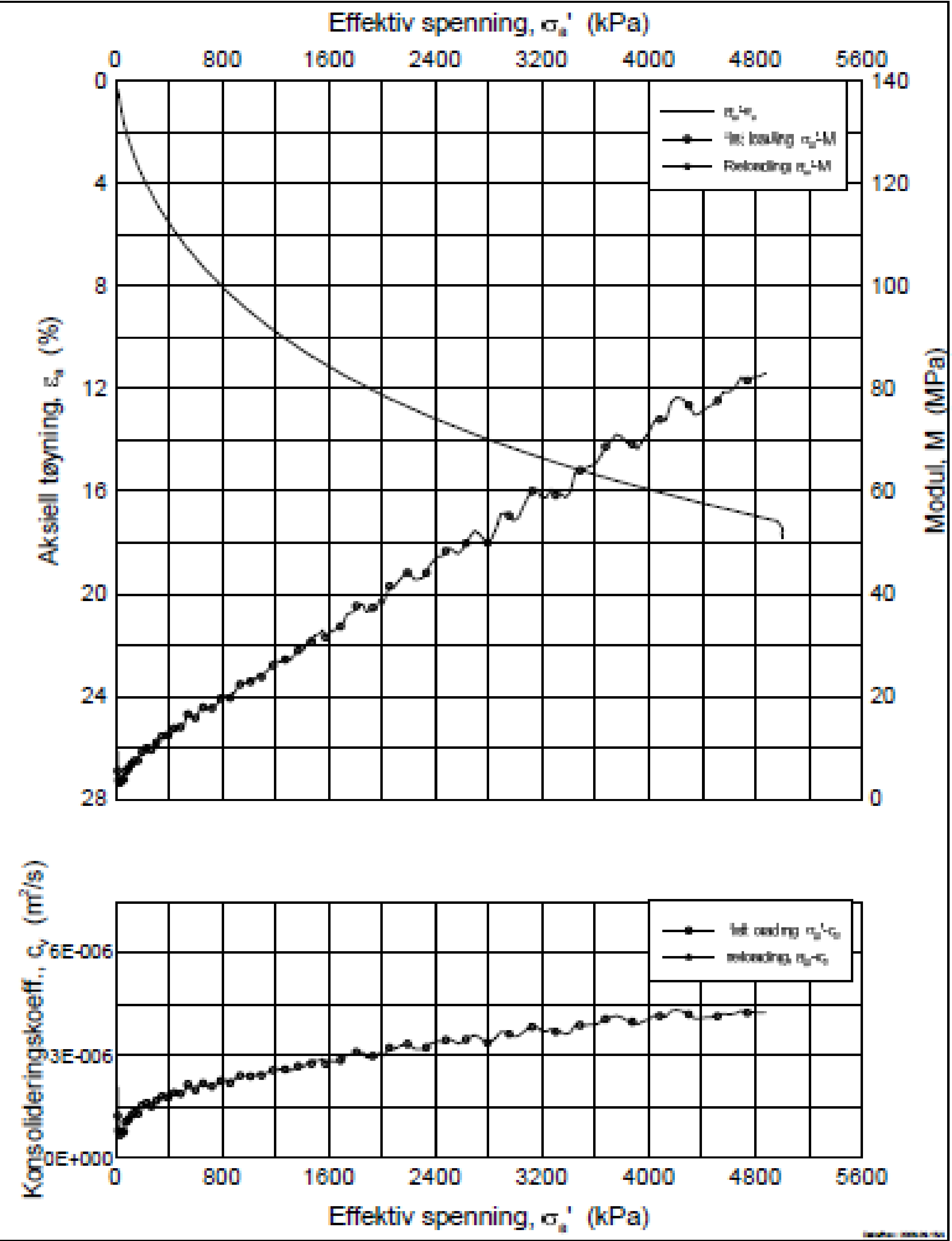
NGI



Sande kommune, kvikkleiresone 502			Dokumentnr: 20110013
Ødometer test (CRSC)		Dybde = 9.30 m	Dato: 2011-11-01
Borhull: 502	Sylinder: 2	p_o' = 93.0 kPa	Figurnr: XX.XX
Del: A	Test: 1	w_i = 32.32 %	Tegner: FPf



Sande kommune, kvikkleiresone 502		Dokumentnr. 20110013
Oedometer test (CRSC)	Dybde = 9.30 m	Dato 2011-11-01
Borhull: 502 Sylinder: 2	p_0' = 93.0 kPa	Figur: XX.XX
Del: A Test: 1	w_i = 32.32 %	Tegner FPf



Sande kommune, kvikkleiresone 502

Dokumentnr.: 20110013

Oedometer test (CRSC)

Dybde = 13.21 m

Dato: 2011-11-01

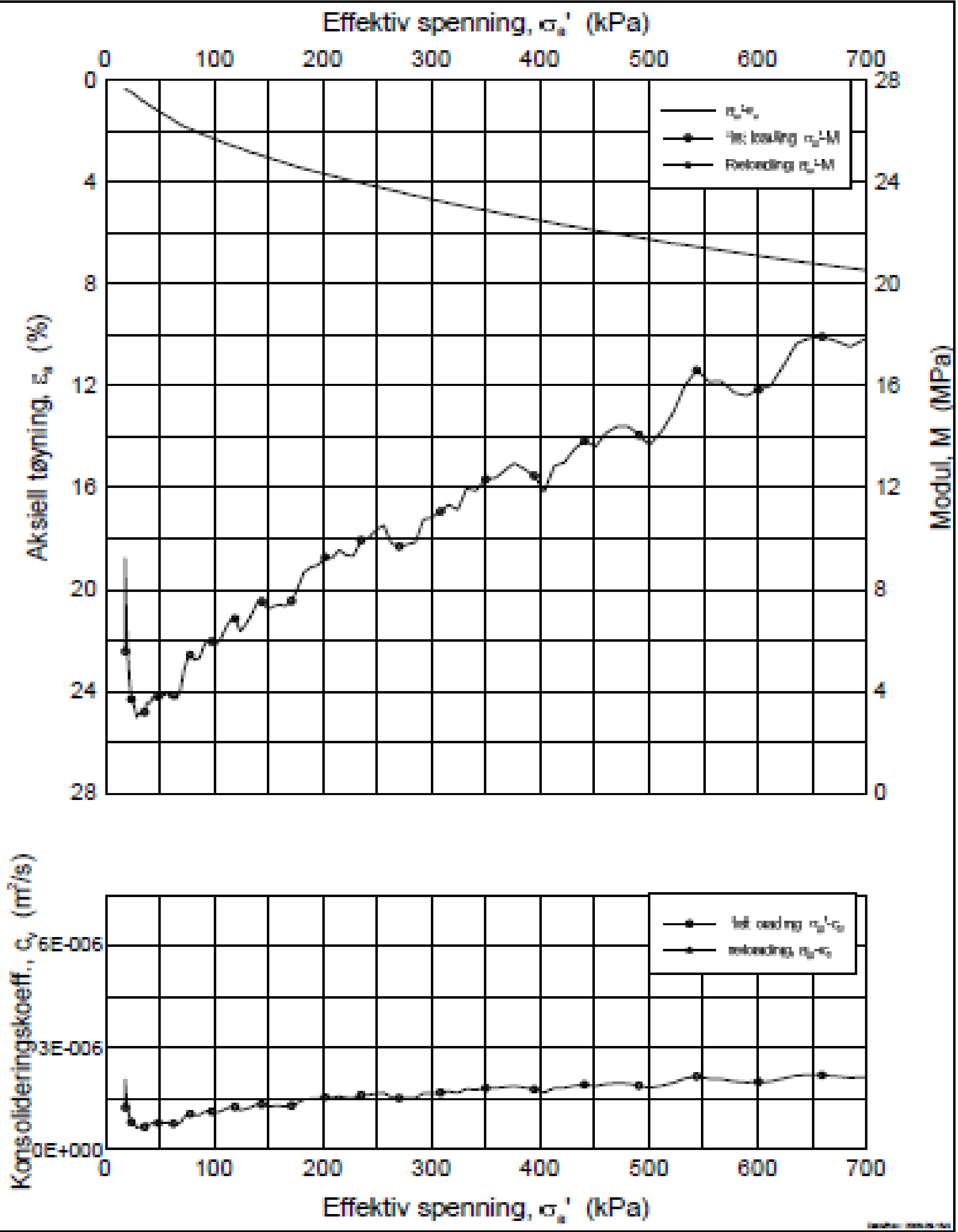
Borhull: 502 Sylinder: 3 p_c' = 132.1 kPa

Figur: XX.XX

Del: A Test: 1 w_i = 33.44 %

Tegner: FPf

NGI



Sande kommune, kvikkleiresone 502

Dokumentnr.: 20110013

Oedometer test (CRSC)

Dybde = 13.21 m

Dato: 2011-11-01

Borhull: 502 Sylinder: 3

p_c' = 132.1 kPa

Figurnr.: XX.XX

Del: A Test: 1

w_i = 33.44 %

Tegner: FPi